جزوه بخش نرم‌افزاری دوره جامع آموزش اصول محاسبات لرزه‌ای و طراحی عملکردی سازه‌ها

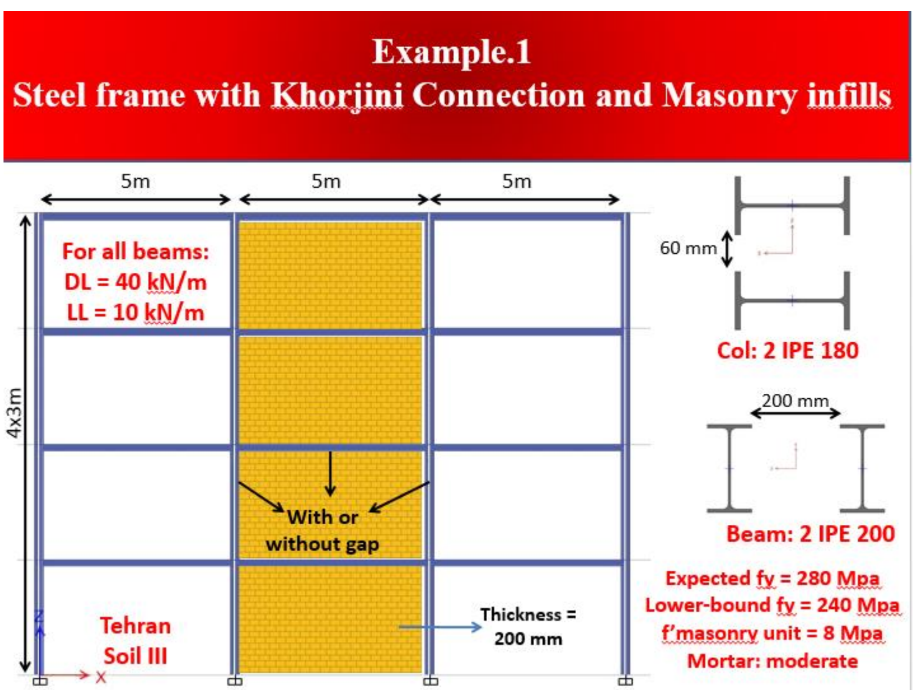
در این بخش از دوره که توسط **دکتر موسوی از مدرسین موسسه ۸۰۸** برگزار می‌شود، به حل تمرینات زیر پرداخته خواهد شد:

۱. قاب ساده خرجینی+ میانقاب آجری

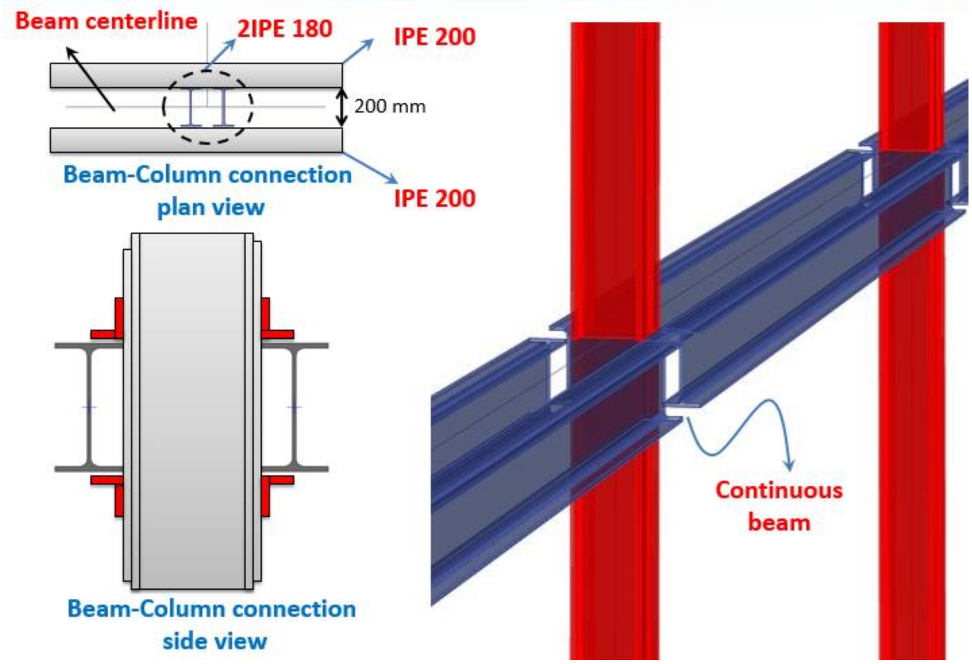
قاب ۴ طبقه سه دهانه فرضیات بار و ابعاد هندسی تا حد امکان مشابه نمونه ۱۰۱۵/ محل بنا در تهران، خاک نوع ۳، دهانه‌ها ۵ متر، ارتفاع همه طبقات ۳ متر، بار مرده و زنده طبقات به ترتیب برابر ۴ و ۱ تن بر متر/ تیرها دوبل آی پی ای ۲۰/ اتصالات خرجینی سنتی با نبشی بالا و پایین و با فرض سختی دوران صفر/ ستون‌ها پا باز دوبل آی پی ای ۱۸/ یک دهانه از سه دهانه دارای میانقاب‌های آجری به کلفتی ۲۰ سانتیمتر در همه طبقات/ مقاومت فشاری آجر ۸ نیوتن بر م م مربع و ملات متوسط.

خروجی‌ها:

۱. جابجایی هدف در خطر ۱ و ۲
۲. مدل میانقاب/ مقاومت، سختی، نمودار غیرخطی
۳. وضعیت لولای خمیری در میانقاب در خطر ۱ و ۲
۴. دوران اتصال خرجینی در خطر ۱ و ۲
۵. نمودار رانش
۶. نمودار دوران اتصال برحسب جابجایی بام
۷. نمودار گریز میانقاب برحسب جابجایی بام



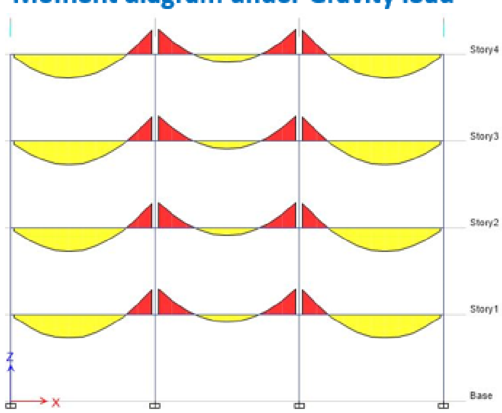
Example.1



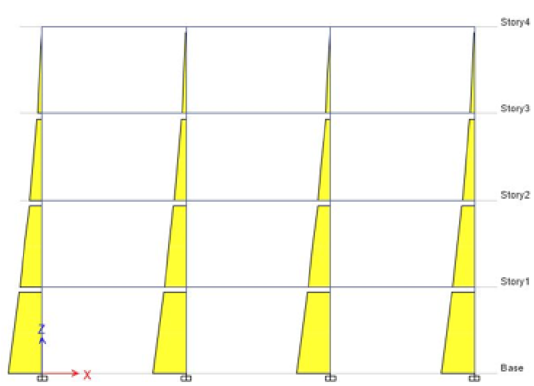
Example.1

Let's assume the bare frame with no infill

Moment diagram under Gravity load



Moment diagram under Lateral load



Bare frame would be unstable in the case of pinned base columns.

The beams are continuous but get no moment from lateral loads .

This can be modeled with a fixed beam-column connection but with a panel zone with zero flexural stiffness.

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Example.1

This type of column sections are not covered in ASCE 41-13 so we use Code 360

Although the beams would not get moment from lateral loads, significant moments could be generated due to the gravity loads as well as the infill. So plastic hinges are defined at the middle of the beams and beam ends.

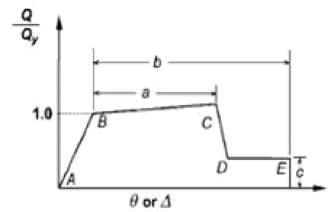


FIG. 9-1. Generalized Force–Deformation Relation for Steel Elements or Components

From ETABS, obtain PCL=1070 kN

From ETABS gravity analysis most column are force-controlled. Still we would define P-M3 Plastic hinges for columns

Define the behavior for P/PCL = 0.2 and assign the parameters for P/PCL<0.2

Define the behavior for P/PCL = 0.45 and assign the parameters for 0.2<P/PCL<0.5

Define the behavior for P/PCL = 0.5 and assign all parameters zero or very small values

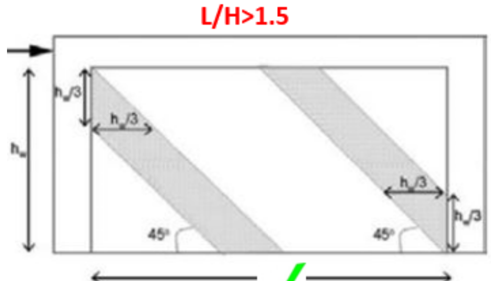
ETABS would automatically interpolate between different P/PCL and also accounts for P/PCL on moment capacity of the column

Per Code 360, rotation of the khorjini connections with top and bottom angles should be limited to 0.01 rad and 0.02 rad for LS and CP criteria, respectively.

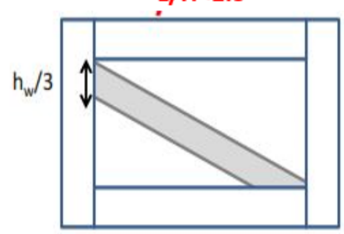
Example.1

Infill modeling per ASCE 41-13

L/H>1.5



L/H<1.5



L/H=5/3=1.67>1.5 ✓

Moderate mortar

f'_b = 8 MPa → f'_m = 2.5 to 2.96 Mpa = 2.7 MPa → Expected f'_m = 1.3x2.7=3.5 MPa

$E_m \approx 400f'_m$=400x3.5=1400 MPa

This is the horizontal component of the ultimate capacity of both struts

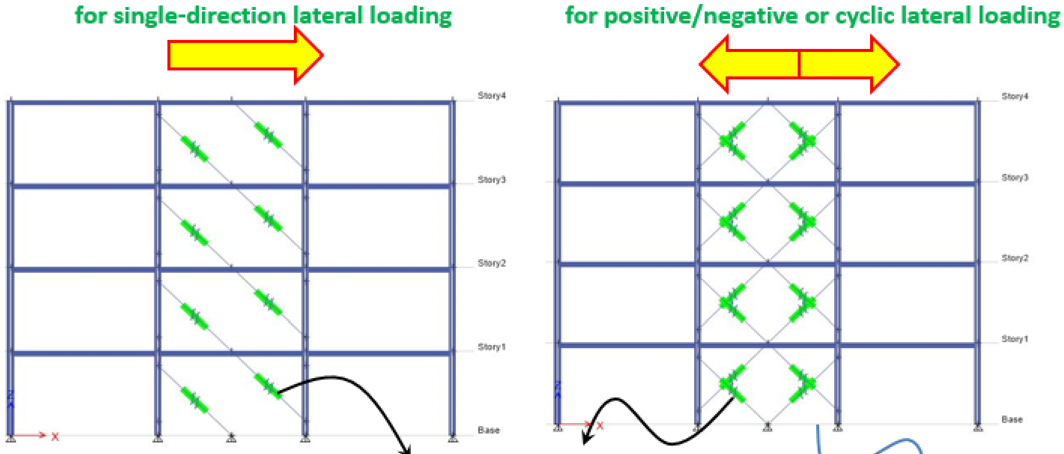
Expected shear capacity: $Q_{CE}=A_n \cdot v_{me\text{-(zero gravity load)}}$=5000(mm)x200(mm)x0.2(Mpa)=200 kN

Should be obtained based on in-situ test: $v_{me\text{-(zero gravity load)}}$=0.2 Mpa (assumed value)

Example.1

for single-direction lateral loading

for positive/negative or cyclic lateral loading



Compression-only Multilinear plastic Link element

In this case, effective stiffness of link elements should be reduced by 50% as there are two struts. In nonlinear analysis only one of them would be active so the initial stiffness should not by reduced.

Length of each strut: L=3.5m

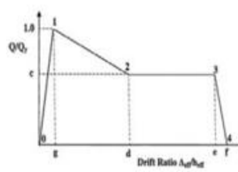
Area of each strut: A=(1/2^0.5)x0.2=0.14 m^2

For each strut:

Initial axial stiffness= EA/L=1400(Mpa)x0.14/3.5=56 kN/mm

Ultimate axial strength =QCE/2/cos(θ)=200/2/cos(45)=141 kN

Example.1



Vfr= sum (2M$_{Pc}$/H$_s$)

Consider C=0
d=f

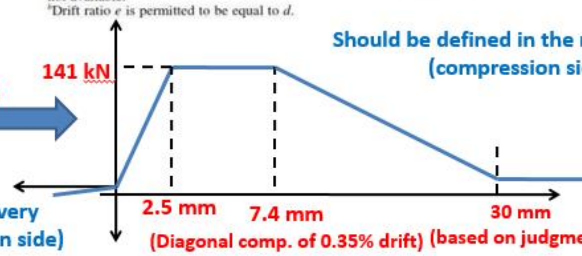
Both ASCE 41-13 and Code 360

Table 11-9. Nonlinear Procedure—Simplified Force–Deflection Relations for Masonry Infill Panels[a]

$\beta = \frac{V_{fre}}{V_{ine}}$	$\frac{L_{inf}}{h_{inf}}$	Residual Strength Ratio c	d (%)	e[b] (%)	Acceptance Criteria LS (%)	Acceptance Criteria CP (%)
$\beta < 0.7$	0.5	NA	0.5	NA	0.4	NA
	1.0	NA	0.4	NA	0.3	NA
	2.0	NA	0.3	NA	0.2	NA
$0.7 \le \beta < 1.3$	0.5	NA	1.0	NA	0.8	NA
	1.0	NA	0.8	NA	0.6	NA
	2.0	NA	0.6	NA	0.4	NA
$\beta \ge 1.3$	0.5	NA	1.5	NA	1.1	NA
	1.0	NA	1.2	NA	0.9	NA
	2.0	NA	0.9	NA	0.7	NA

[a]Interpolation shall be used between table values. In this table, NA means not available.
[b]Drift ratio e is permitted to be equal to d.

In this example: Vf=0
L/H=1.67



Should be defined in the negative side (compression side)

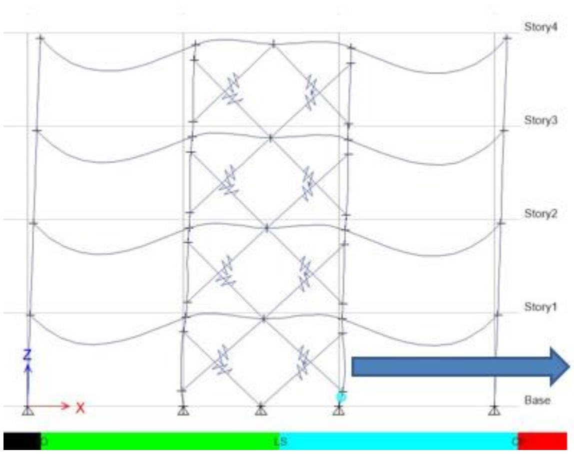
Linear elastic with very small stiffness (tension side)

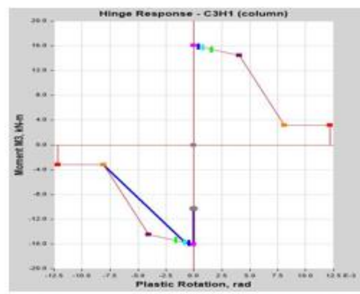
(Diagonal comp. of 0.35% drift) (based on judgment)

Example.1

Per Code 360
Seismic mass: D+0.2L
Gravity load: 1.1(D+0.25L)

It turned out that the structure is too weak even for gravity loads

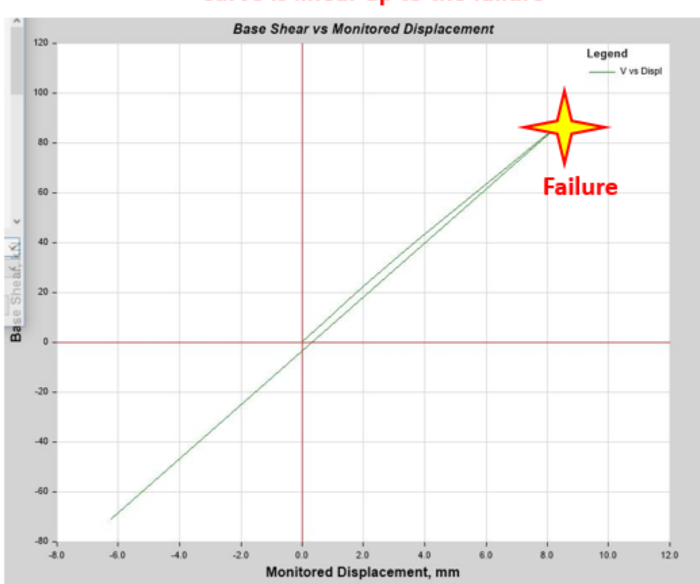




Under gravity load, P/PCL ≈0.85
At roof disp. Of only 8 mm, P/PCL ≈1

Example.1

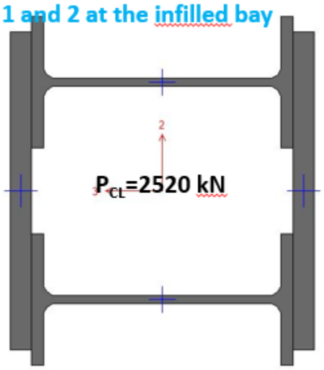
The column failed before infills and the pushover curve is linear up to the failure



Example.1

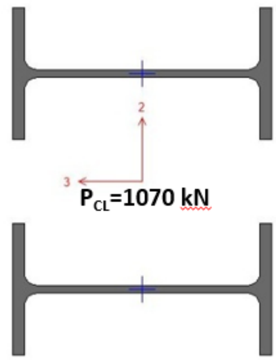
So it is decided to improve columns at the lower two stories but it is not intended to fully retrofit the frame (that is, columns are just moderately improved)

Column section for Stories 1 and 2 at the infilled bay



2 IPE 180 + 2PL220x15

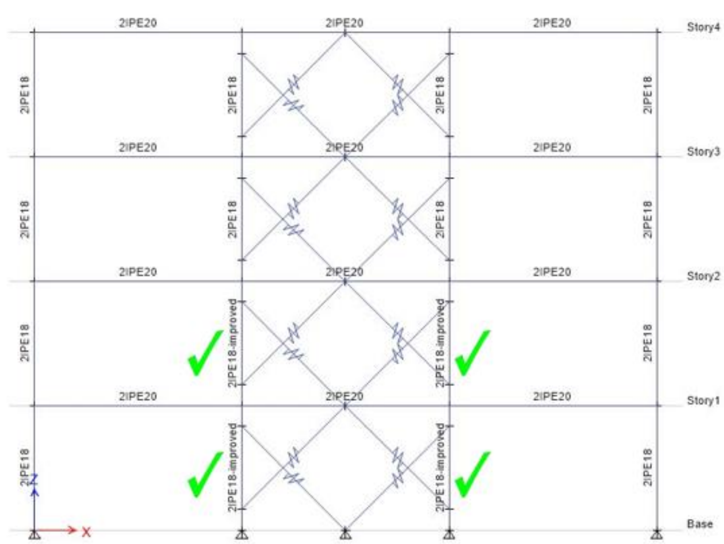
Other columns



2 IPE 180

Example.1

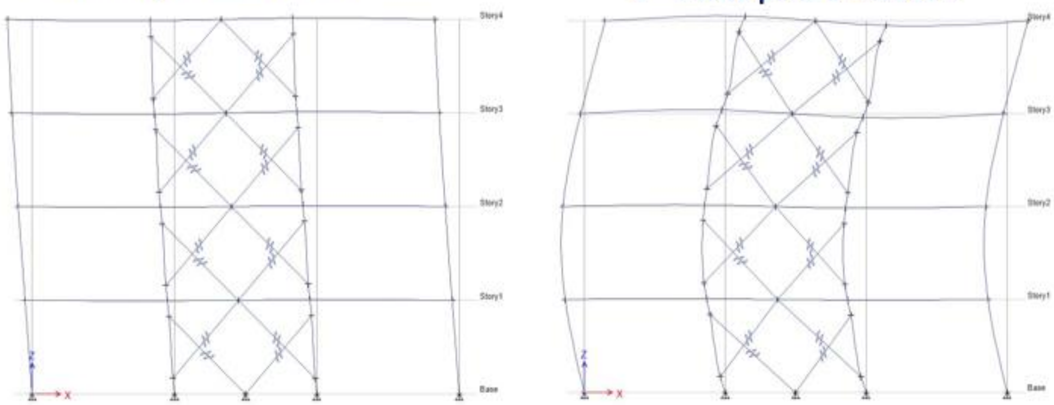
Improved columns



Example.1

1st mode period = 0.845 s

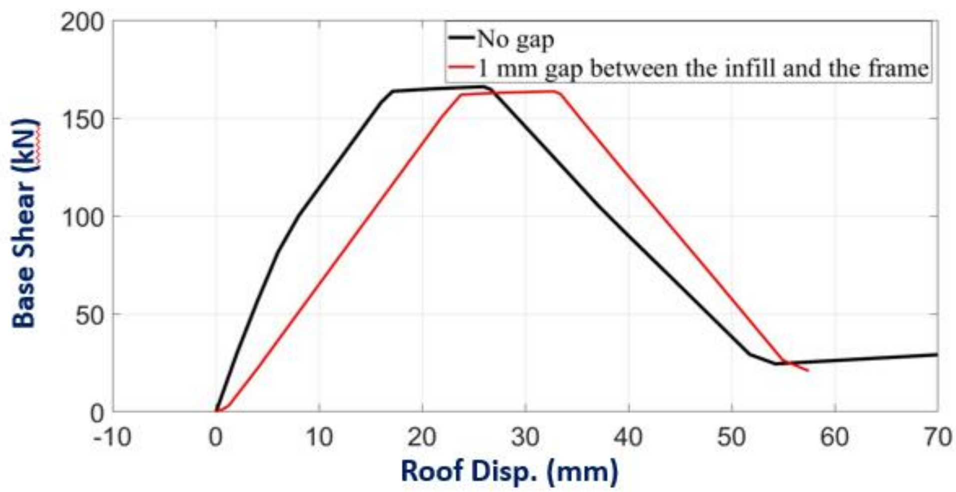
2nd mode period = 0.289 s



Example.1

We can define the gap between infill and the frame as well. Considering a small gap (1 to 5mm) is recommended to avoid unrealistic axial loads on the struts during gravity loading

Example.1



Example.1

Level I Seismic hazard (return period of 475 years or 10%-50 years)

Target disp. = 226 mm

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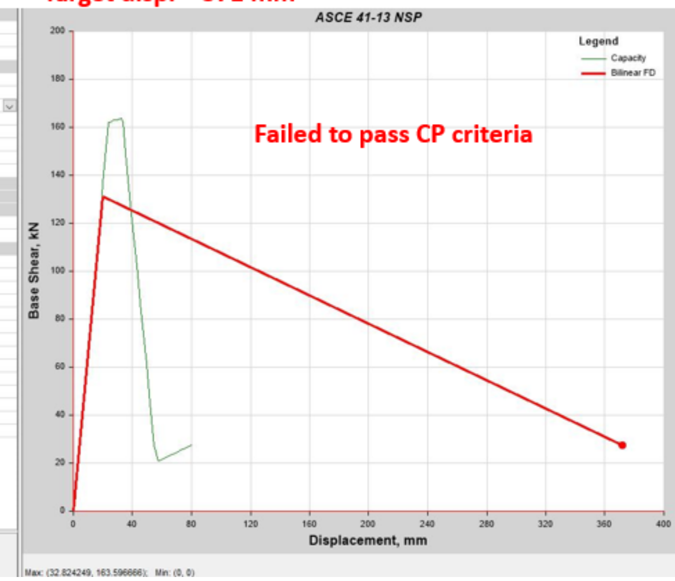
Example.1

Level II Seismic hazard (return period of 2500 years or 2%-50 years)

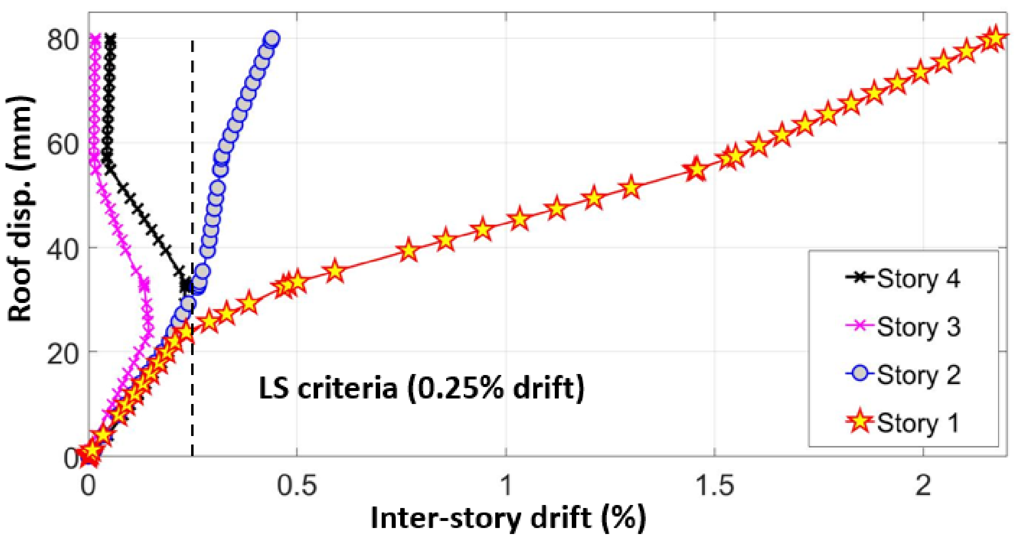
Target disp. = 371 mm

Plot Definition	
Plot Type	ASCE 41-13 NSP
Load Case	push
Legend Type	Integrated
Demand Spectrum	
Damping Ratio	0.05
Spectrum Source	Defined Function
Function Name	st2800
SF (mm/sec²)	15000
Ts (sec)	1
Include SSI	No
C2 Type	Default Value
Cm Type	Default Value
Capacity Curve	
Bilinear Force-Displacement Curve	
Target Displacement Results	
Displ. (mm)	371.884
Shear (kN)	27.5513
Calculated Parameters	
C0	1.025866
C1	1.382427
C2	1
Sa, g	1.131983
Te (sec)	0.961
Ki (kN/m)	2045.376
Ke (kN/m)	6671.776
Ti (sec)	1.735
Alpha	-0.044049
uStrength	22.186683
Dy (mm)	19.645
Vy (kN)	131.0689
Weight (kN)	2568.9287
Cm	1

Function Name
The defined response spectrum function name.



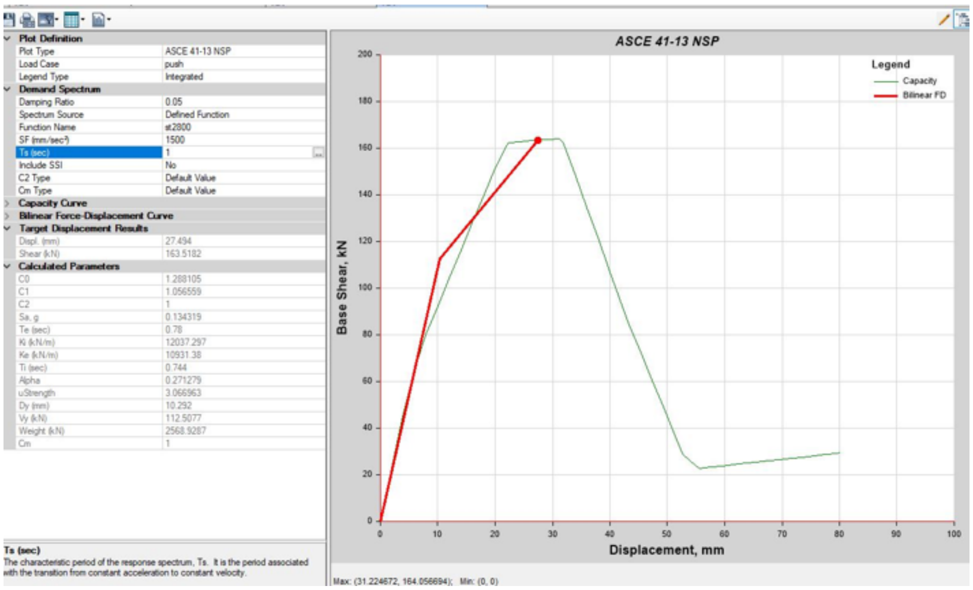
Example.1



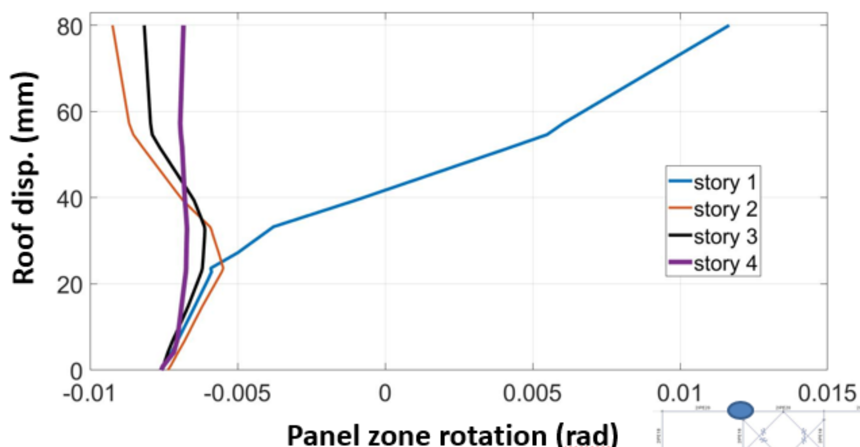
After roof disp. Of about 30 mm , significant deviation has occurred between drift of the first story and that of other stories indicating that a soft story mechanism would occur once the frame experience a roof displacement of more than about 30 mm

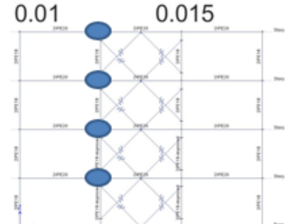
Example.1

This frame can only sustain (in LS limit) an earthquake 6.5 times smaller than the design earthquake



Example.1





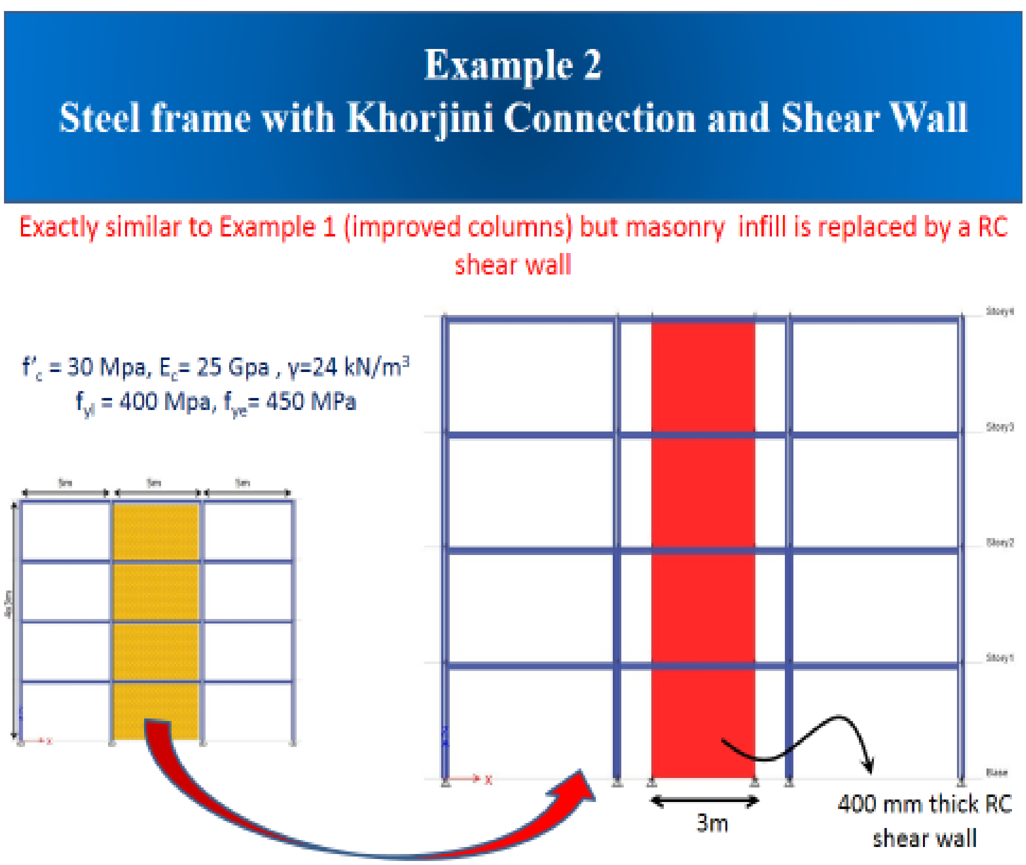
Panel zone rotation of this kind of Khorjini connection should be less than 0.01 rad for LS and 0.02 rad for CP criteria

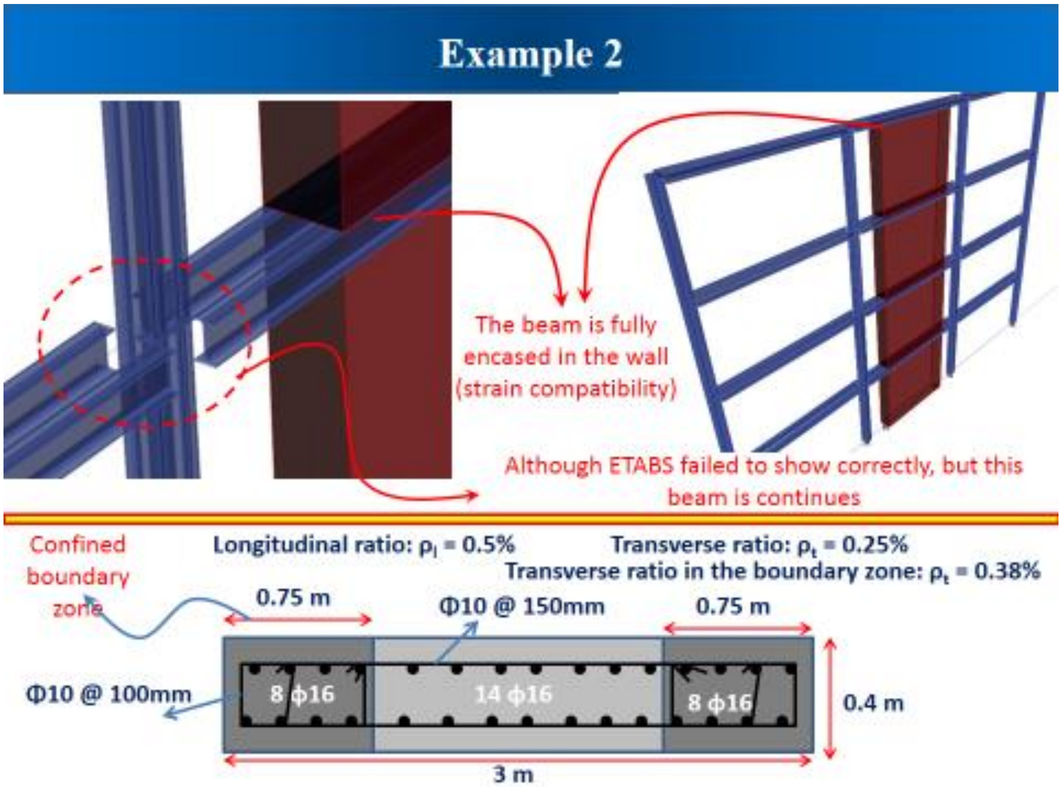
۲. قاب ساده خرجینی + دیوار برشی

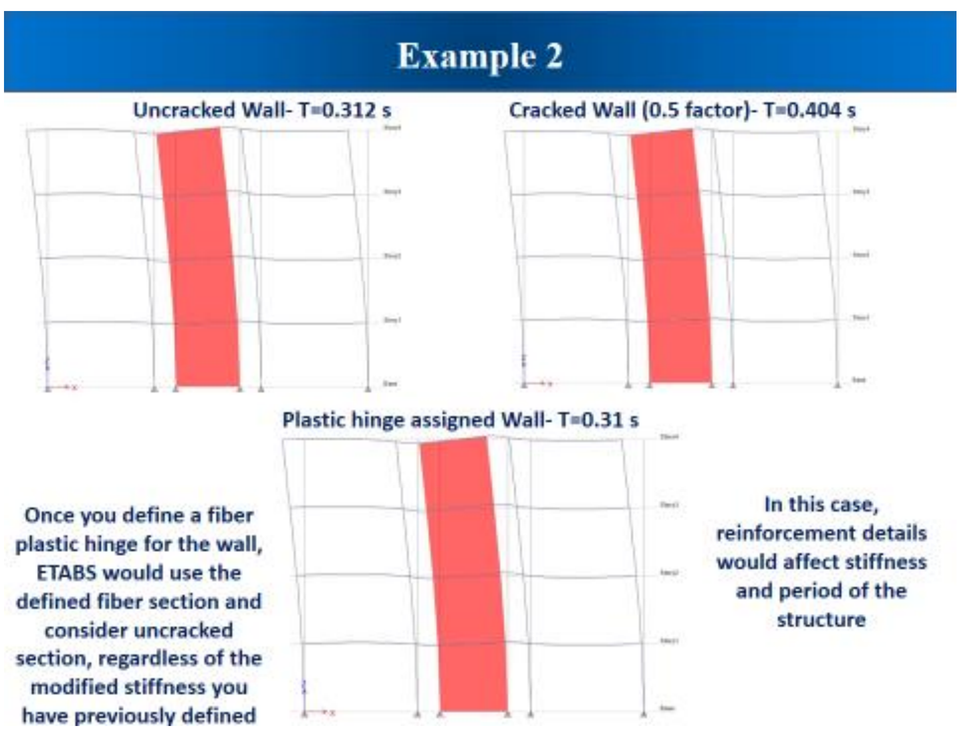
مشابه حالت ۱/ بجای میانقاب از یک دیوار برشی لبه‌دار به ضخامت ۰/۴، طول ۳ متر و میلگرد طولی ۰/۵٪ استفاده شود.

خروجی‌ها:

۱. جابجایی هدف در خطر ۱ و ۲
۲. مدل دیوار برشی/ مقاومت‌های برشی و خمشی، سختی، نمودار غیرخطی
۳. وضعیت لولای خمیری در دیوار برشی در خطر ۱ و ۲: لولای خمشی یا برشی؟
۴. دوران دیوار برشی در خطر ۱ و ۲
۵. نمودار رانش
۶. نمودار دوران اتصال برحسب جابجایی بام
۷. نمودار لولای خمشی در دیوار برشی برحسب جابجایی بام
۸. درصورتی‌که دیوار برشی بر روی دو شمع تکیه کرده باشد نیروهای فشاری و کششی در این شمع‌ها در حالت اوج




Example 2
The beam is fully encased in the wall (strain compatibility)
Although ETABS failed to show correctly, but this beam is continues
Confined boundary zone
Longitudinal ratio: ρl = 0.5%
Transverse ratio: ρt = 0.25%
Transverse ratio in the boundary zone: ρt = 0.38%
0.75 m
Φ10 @ 150mm
0.75 m
Φ10 @ 100mm
8 ϕ16
14 ϕ16
8 ϕ16
0.4 m
3 m


Example 2
Uncracked Wall- T=0.312 s
Cracked Wall (0.5 factor)- T=0.404 s
Plastic hinge assigned Wall- T=0.31 s
Once you define a fiber plastic hinge for the wall, ETABS would use the defined fiber section and consider uncracked section, regardless of the modified stiffness you have previously defined
In this case, reinforcement details would affect stiffness and period of the structure

Example 2

Very important: Plastic hinges can be developed in any section along the beam length, especially in the case of unconventional structures like the one we are considering

We consider two case:

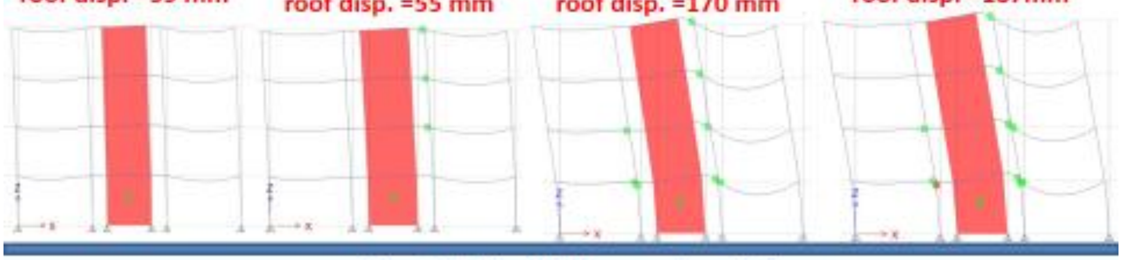
Case 1 (wrong assumption): plastic hinges can be developed only at beam ends

Case 2 (rational assumption): plastic hinges can be developed not only at the ends but also at the middle of the beam and just before wall-beam connection

Example 2

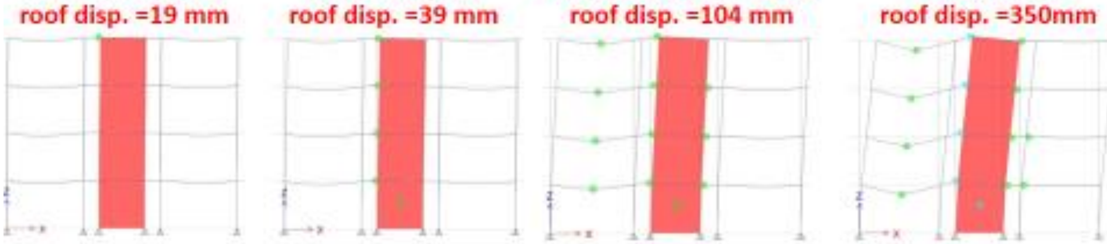
Case 1: 2 plastic hinge at beam ends

First wall yield roof disp. =39 mm	First beam yield roof disp. =55 mm	First column yield roof disp. =170 mm	Final damage roof disp. =187mm

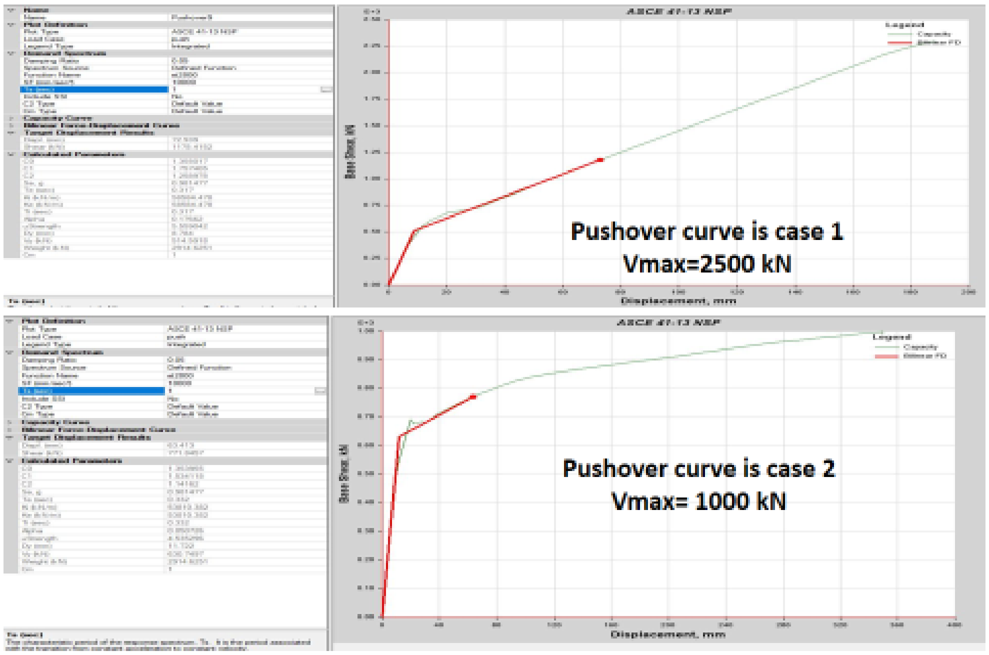


Case 2: 5 plastic hinges at each beam

First beam yield roof disp. =19 mm	First wall yield roof disp. =39 mm	First middle beam yield roof disp. =104 mm	Final damage roof disp. =350mm



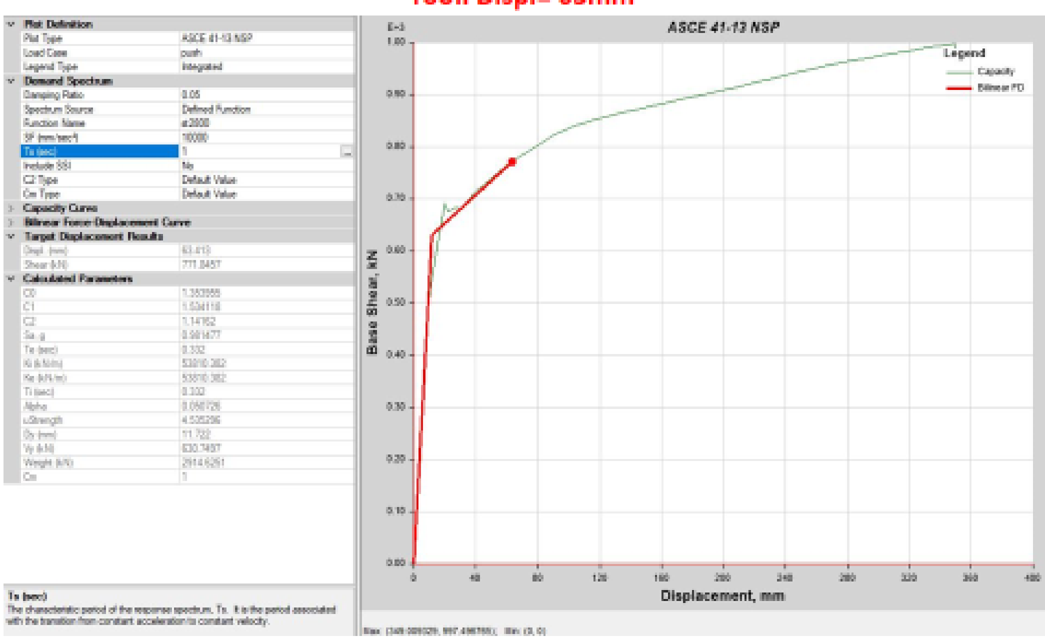
Example 2



Example 2

Under Seismic hazard I

roof. Disp.= 63mm



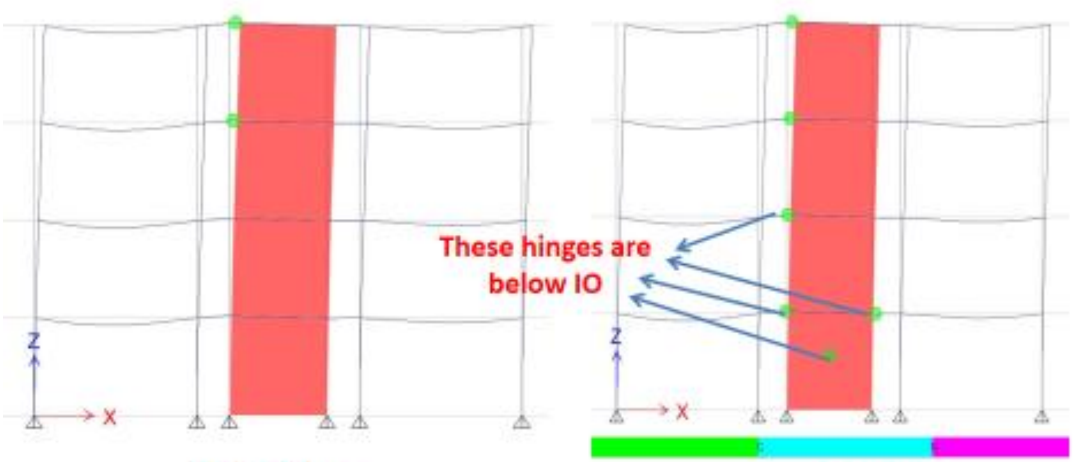
Example 2

So wrong plastic hinge placement can lead to significantly misleading results

Hereafter, let's consider only the true model (case 2 with distributed plasticity along the beams)

Example 2

Under Seismic hazard I
roof. Disp.= 63mm



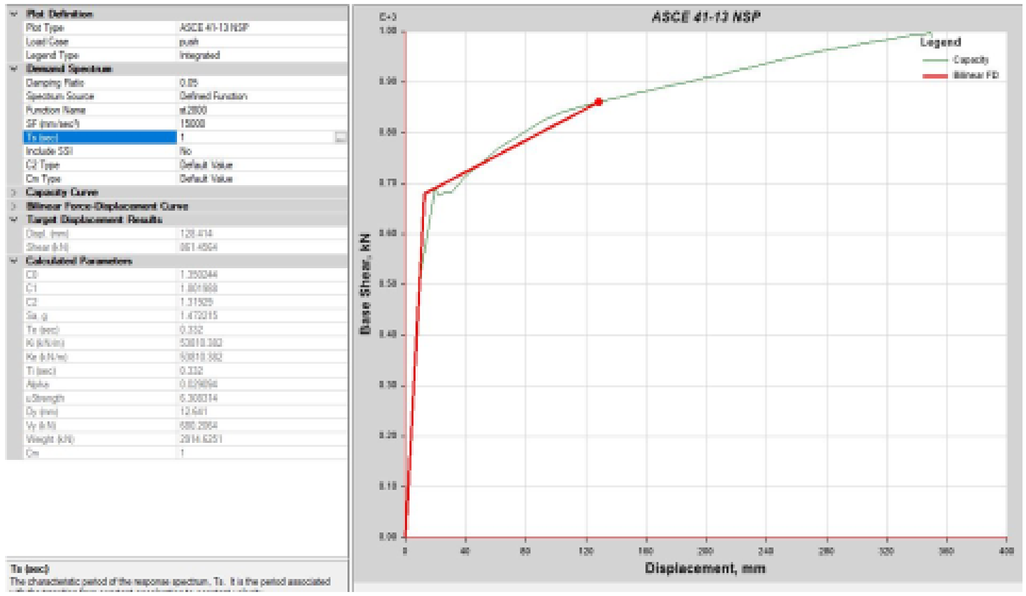
IO to LS hinges

B to C

The structure satisfied the LS criteria under Seismic hazard level I

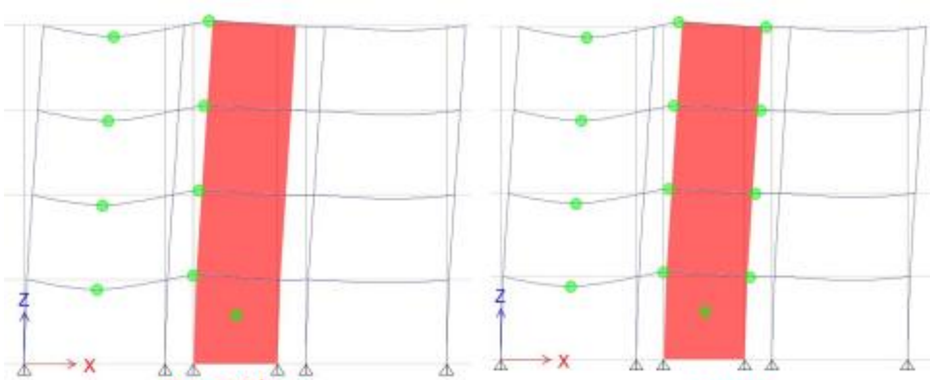
Example 2

Under Seismic hazard II
roof. Disp.= 128mm



Example 2

Under Seismic hazard II
roof. Disp.= 128mm

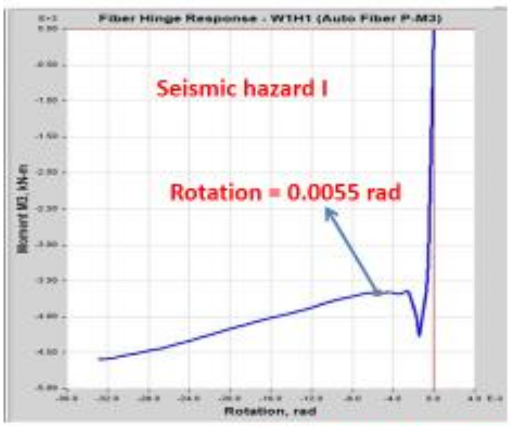


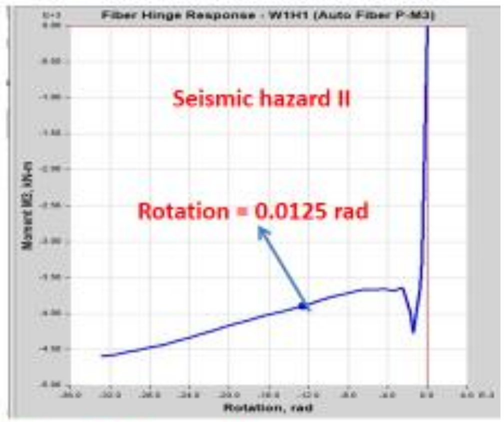
IO to LS hinges **B to C**

The structure satisfied not only the CP but also the LS criteria under Seismic hazard level II

Example 2

Wall plastic hinge





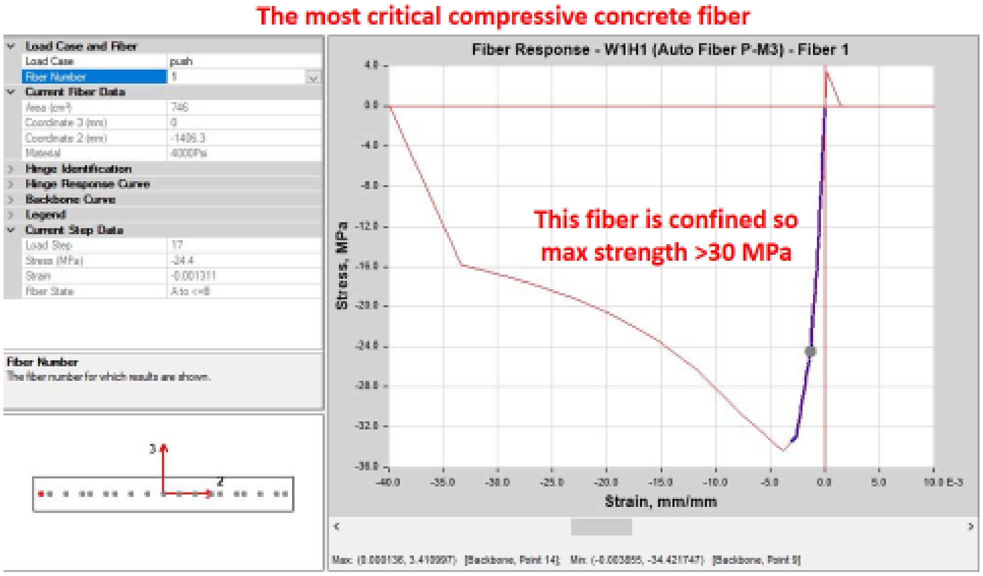
ASCE 41-13's Table 10-19 for flexural RC shear walls

Conditions			Plastic Hinge Rotation (radians) a	b	Strength Ratio c	Performance Level IO	LS	CP
i. Shear walls and wall segments								
$\frac{(A_s - A'_s) f_y + P}{t_w l_w f'_c}$	$\frac{V}{t_w l_w \sqrt{f'_c}}$	Confined Boundary[b]	0.015					
≤0.1	≤4	Yes	0.010	0.020	0.75	0.005	0.015	0.020

Example 2

Seismic hazard II = Step 17

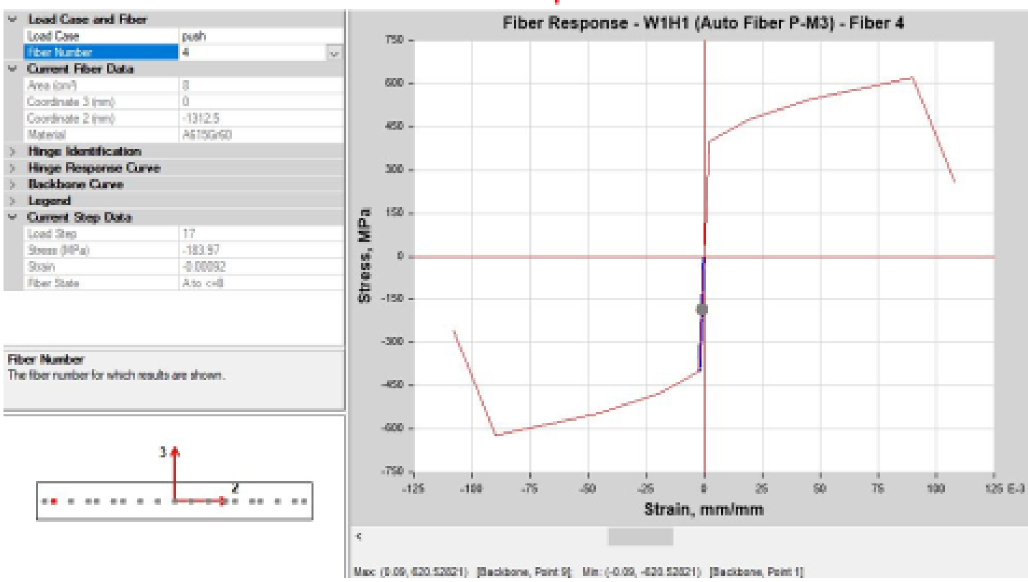
The most critical compressive concrete fiber



Example 2

Seismic hazard II = Step 17

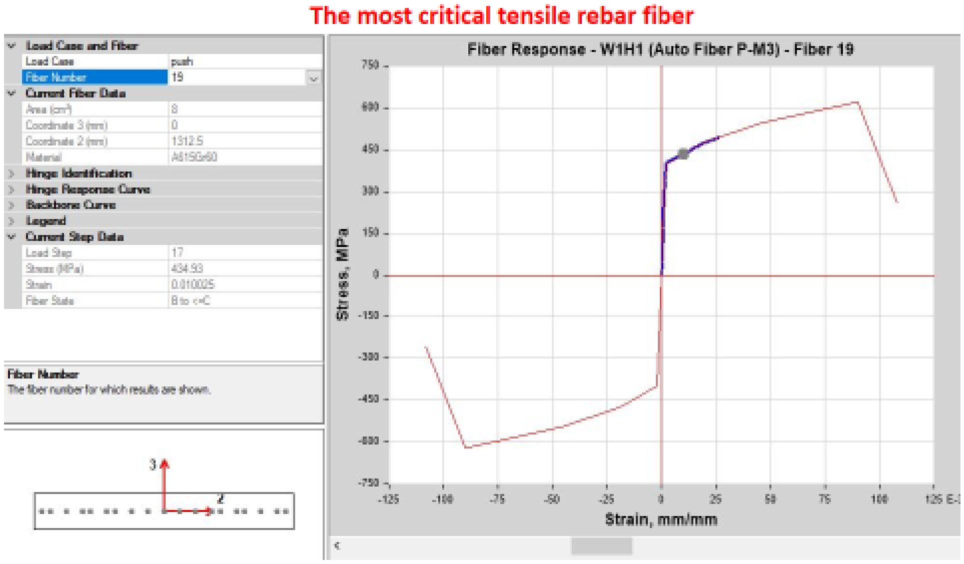
The most critical compressive rebar fiber



Example 2

Seismic hazard II = Step 17

The most critical tensile rebar fiber



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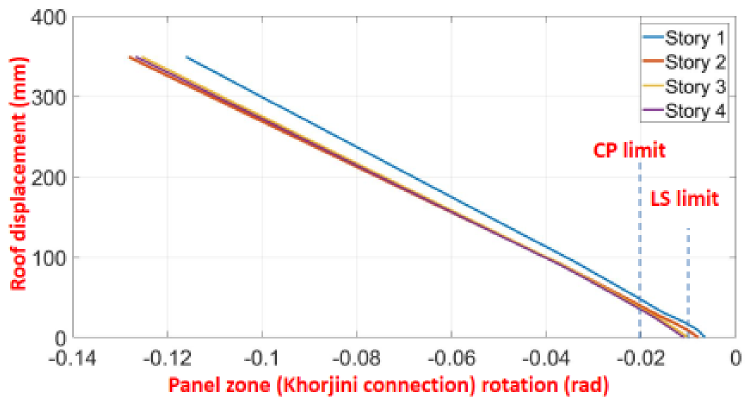
آموزشهای تخصصی عمران و معماری

Example 2

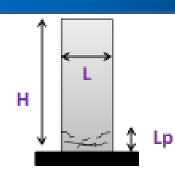
ETABS use strain based criteria to define IO, LS, and CP criteria
The results are not necessarily the same as ASCE 41's or Code 360's Tables which are based on plastic rotations. But in the case of well detailed flexural walls the results are rather in agreement.

Example 2

So Khorjini connections cannot sustain the imposed rotations. However, these connections are secondary component and their failure doe not mean lateral instability. However, stability under gravity loads should be assessed.



Example 2



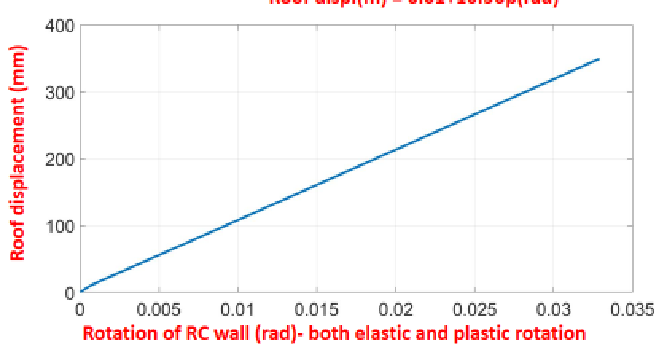
Roof disp. = θyH/3+θp(H-0.5Lp)

Lp = min (0.5 L, story height)

LP = 1.5 m

$\theta_y = \left(\frac{M_y}{E_c I}\right) l_p$ = 0.0025 rad

Roof disp.(m) = 0.01+10.5θp(rad)

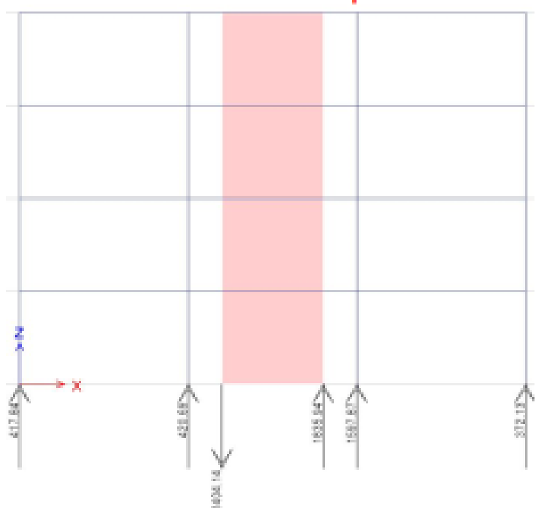


Example 2

Axial demands on the piles at the two edges of the wall

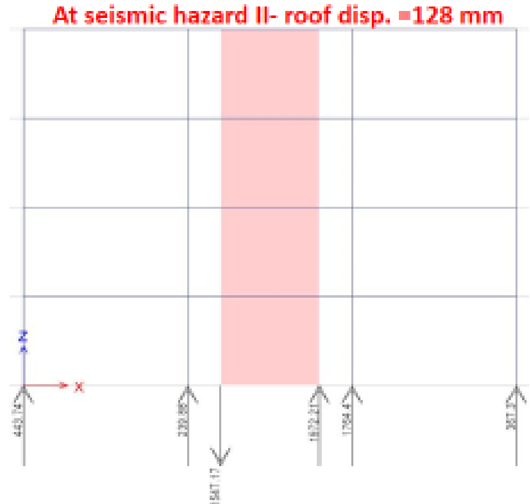
SSI is not considered in this example. If SSI was considered, these demands would be different, probably smaller.

At seismic hazard I- roof disp. =63 mm



Pt = 1404 kN Pc = 1836 kN

At seismic hazard II- roof disp. =128 mm



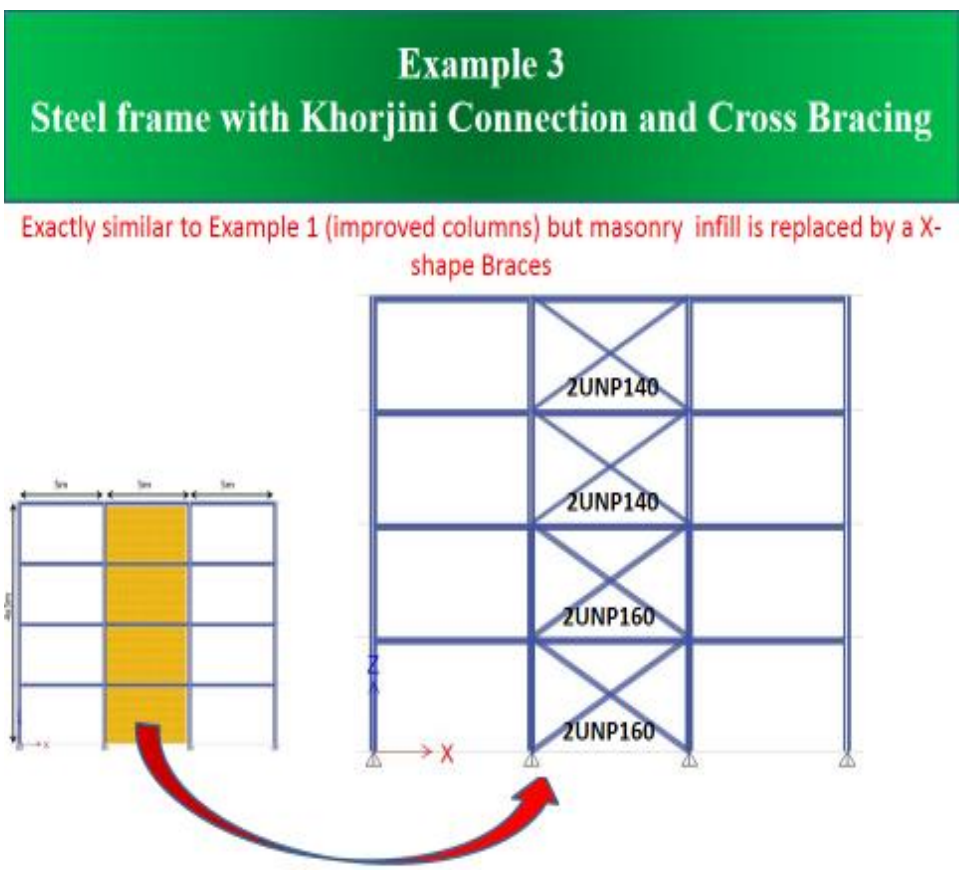
Pt = 1547 kN Pc = 1972 kN

۳. قاب ساده خرجینی + مهاربند ضربدری

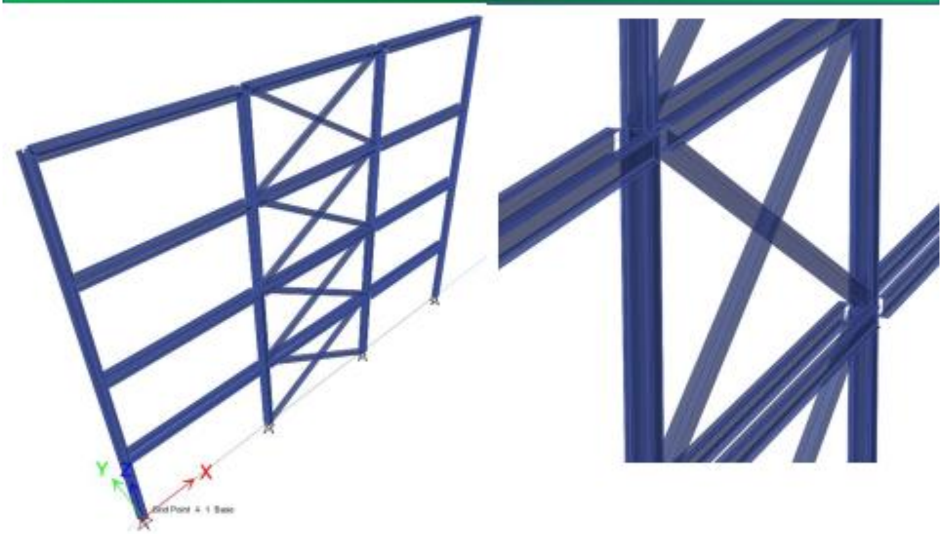
مشابه حالت ۱/ بجای میانقاب از مهاربند ضربدری/ دوبل ناودانی ۱۶ در دو طبقه اول و دوبل ناودانی ۱۴ در طبقات ۳ و ۴.

خروجی‌ها:

۱. جابجایی هدف در خطر ۱ و ۲
۲. مدل مهاربندهای کششی و فشاری / مقاومت‌های فشاری و کششی، نمودار غیرخطی
۳. وضعیت لولای خمیری در مهاربندهای فشاری و کششی در خطر ۱ و ۲
۴. تغییر شکل محوری مهاربندهای کششی و فشاری در خطر ۱ و ۲
۵. نمودار رانش
۶. نمودار دوران اتصال خرجینی برحسب جابجایی بام
۷. نمودار تغییر شکل محوری مهاربندهای کششی و فشاری برحسب جابجایی بام
۸. نمودار نیروی محوری ستون‌های مجاور مهاربند در طبقات اول و دوم



Example 3



Exactly similar to Example 1 (improved columns) but masonry infill is replaced by a X-shape Braces

Example 3

Table B.4.1 (AISC 360-05)

b/t	$1.12\sqrt{E/F_y}$	$1.40\sqrt{E/F_y}$
	compact	noncompact

Table D.1.1 (AISC 341-10)

b/t	$0.55\sqrt{E/F_y}$ [a]	$0.64\sqrt{E/F_y}$ [c]
	Highly ductile member	Moderately ductile member

[a]Section compactness: Acceptance criteria applies to brace sections that are concrete-filled or seismically compact according to Table D1.1 of AISC 341 for highly ductile members. Where the brace section is noncompact according to Table B4.1 of AISC 360, the acceptance criteria shall be multiplied by 0.5. For intermediate compactness conditions, the acceptance criteria shall be multiplied by a value determined by linear interpolation between the seismically compact and the noncompact cases.

If b/t < highly ductile compactness → Use the defined acceptance criteria in the ASCE 41 Table

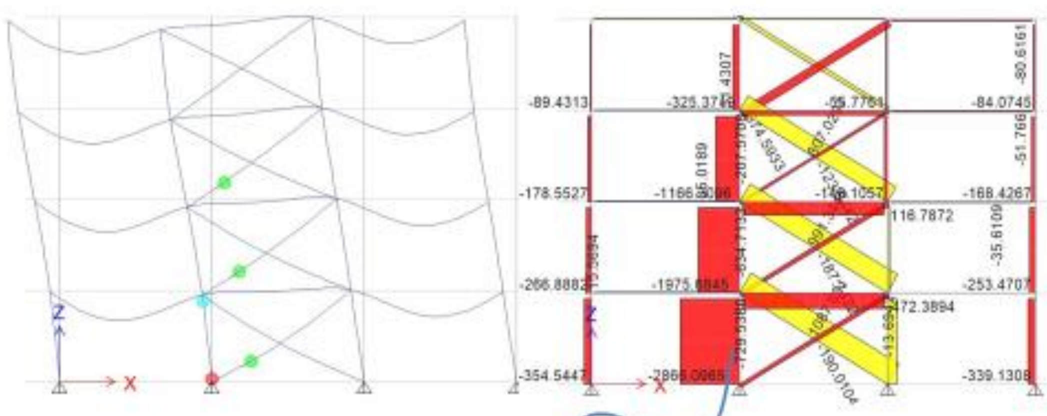
If b/t > compact → Multiply the above criteria by 0.5

For b/t in between above limits → Use interpolation

For poorly detailed gusset plates, acceptance criteria should be multiplied by 0.8

Example 3

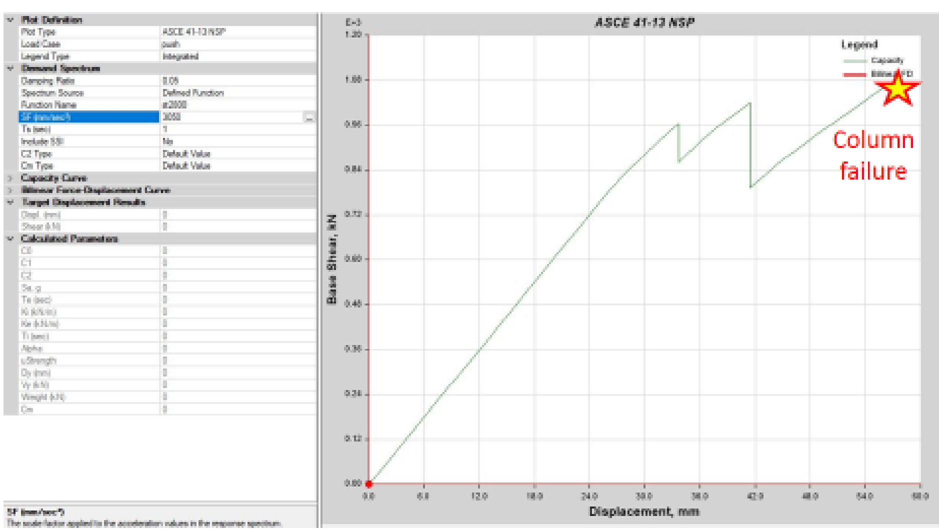
The column failed after only roof disp. Of 40 mm



Axial load = 2866 kN
Axial lower-bound capacity = PCL=2520 kN

Example 3

Pushover curve
This frame can sustain only 30% of level I seismic hazard. i.e. an earthquake with a return period of about 10 years



Example 3

Let's consider weaker braces

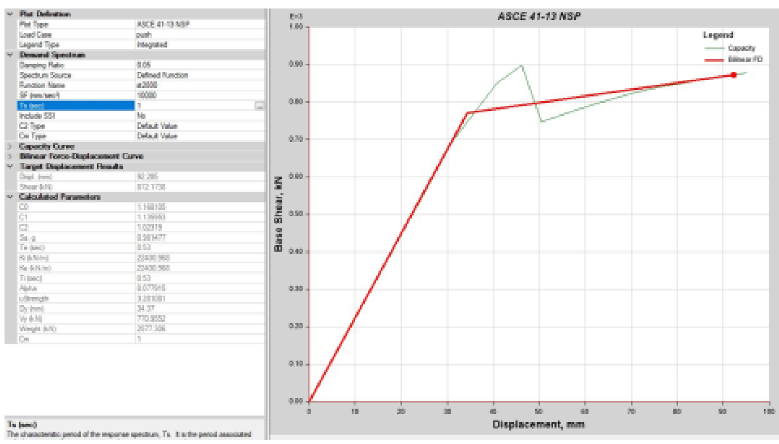
2UNP100 for 1st story and 2UNP80 for other stories

Now, the columns can sustain additional axial demands and would fail after the braces

Example 3

Seismic hazard I

Target disp. = 92 mm

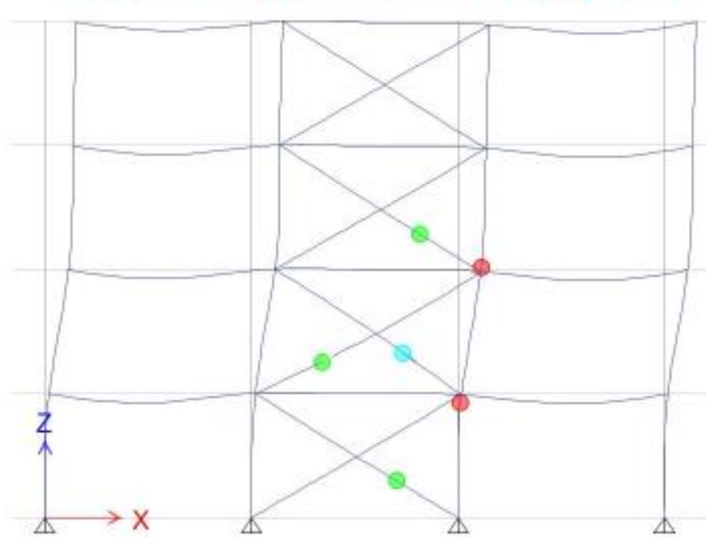


Example 3

Seismic hazard I

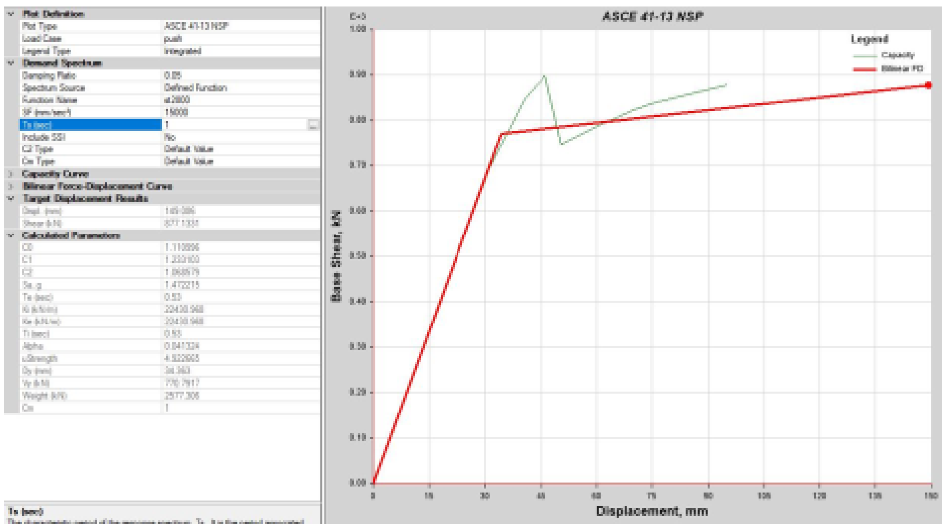
Target disp. = 92 mm

Some of column and braces cannot satisfy LS criteria



Example 3

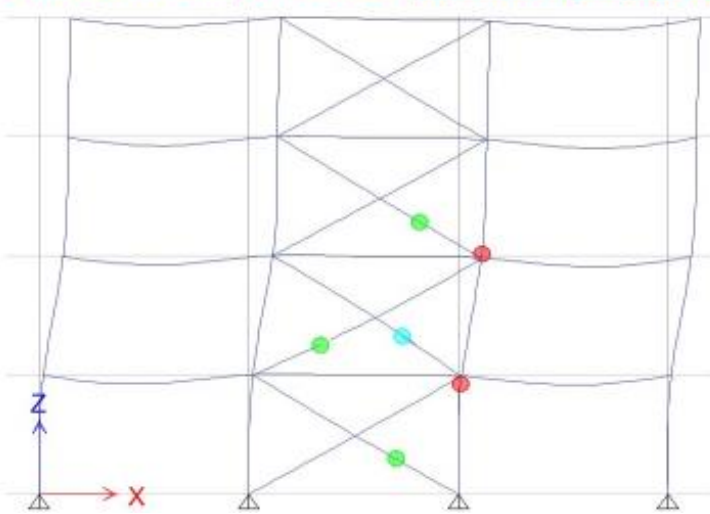
Seismic hazard II
Target disp. = 149 mm



Example 3

Seismic hazard II
Target disp. = 149 mm

The frame cannot satisfy CP criteria….*it failed at roof disp. of 95 mm*



Example 3

Brace plastic hinges are user-defined

1. Obtained tensile and compressive strength of the braces (from design capability of the program). Consider unbraced length modifications.

2. Increase obtained strength due to expected yield strength as well as unit strength reduction factor. For example, if the tensile strength from design is 500 kN. And expected to specified yield strength is 1.1, then the expected tensile strength would be 500x1.1/0.9=611 kN (0.9 is the reduction factor)

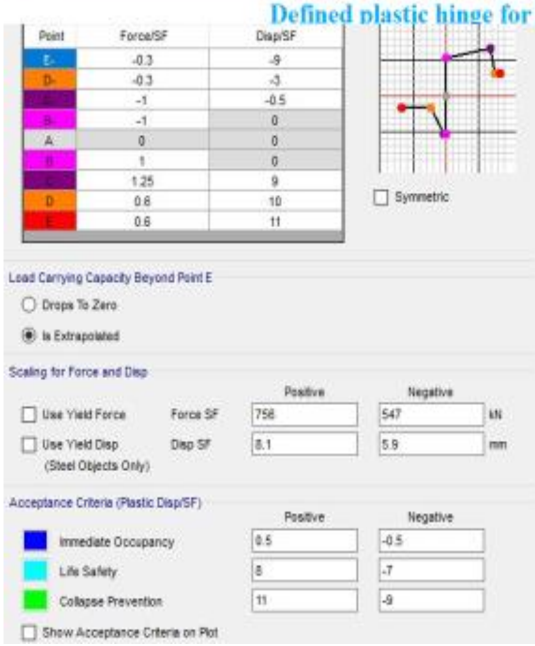
3. Obtain axial stiffness of the brace (EA/L)

4. Obtain tensile and compressive yield displacement

5. Input the strength and yield displacement in the Table of the plastic hinge definition

Example 3

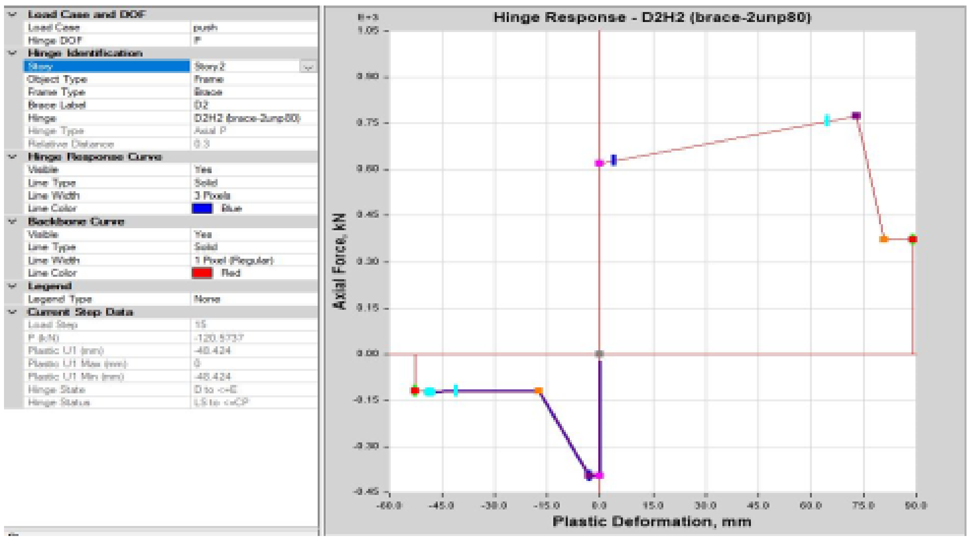
Defined plastic hinge for 2UNP100



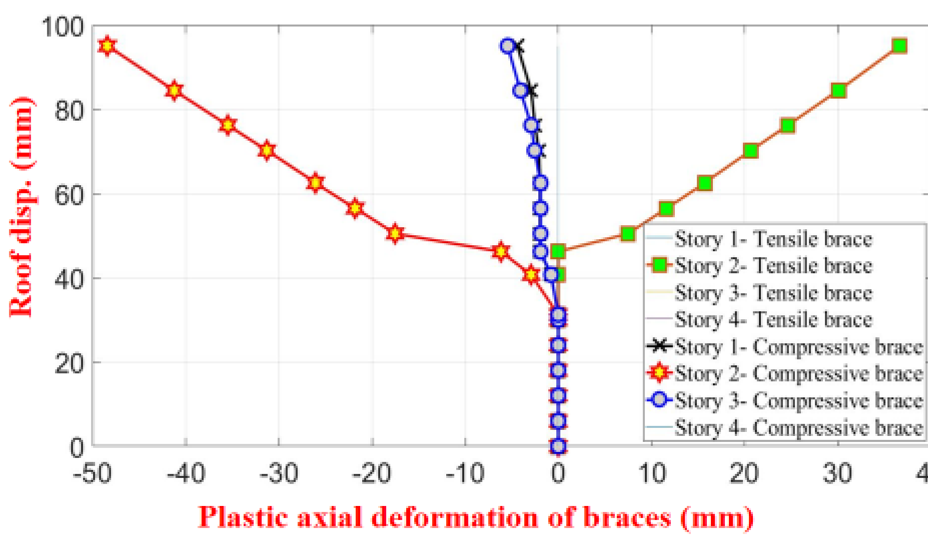
If you are using auto hinge, always check the generated hinges. Sometimes sign of the values on the table would not correctly generated.

Example 3

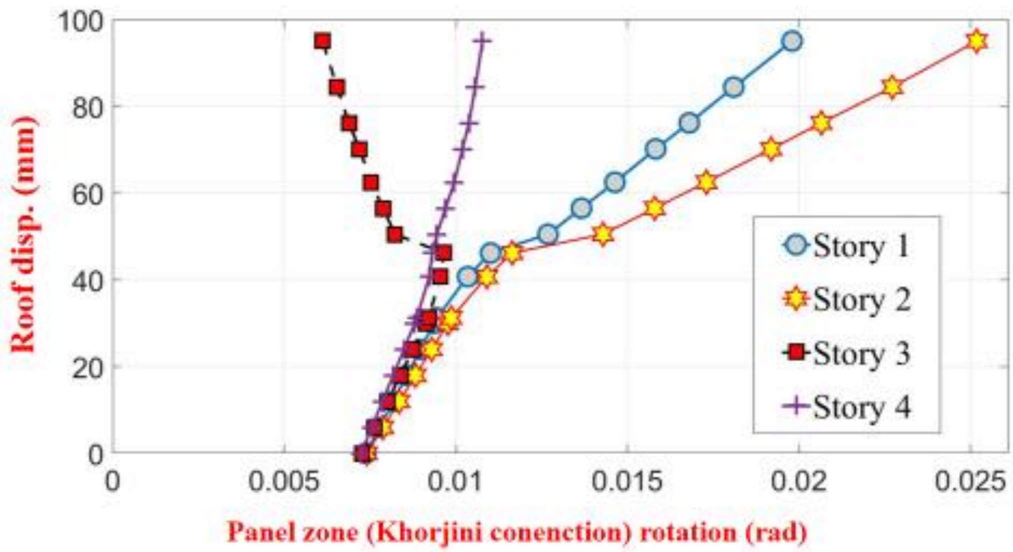
Plastic hinge of the 2nd story brace (2UNP80)



Example 3

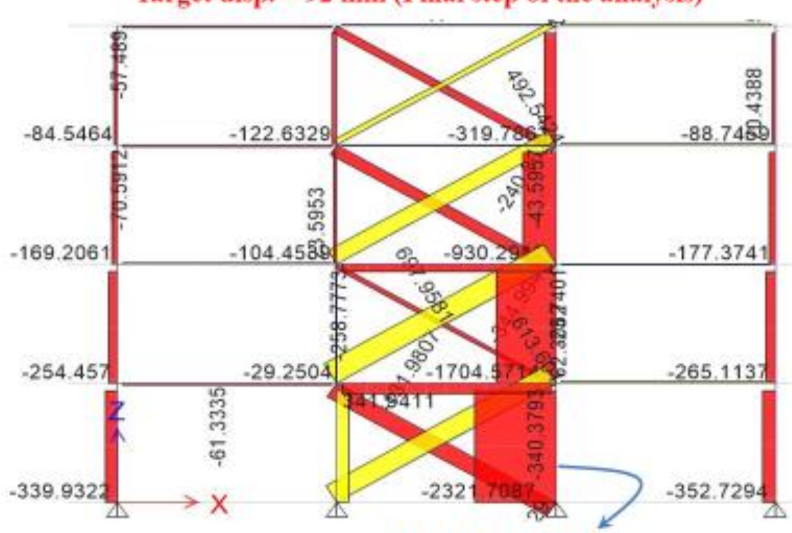


Example 3



Example 3

Seismic hazard I
Target disp. = 92 mm (Final step of the analysis)



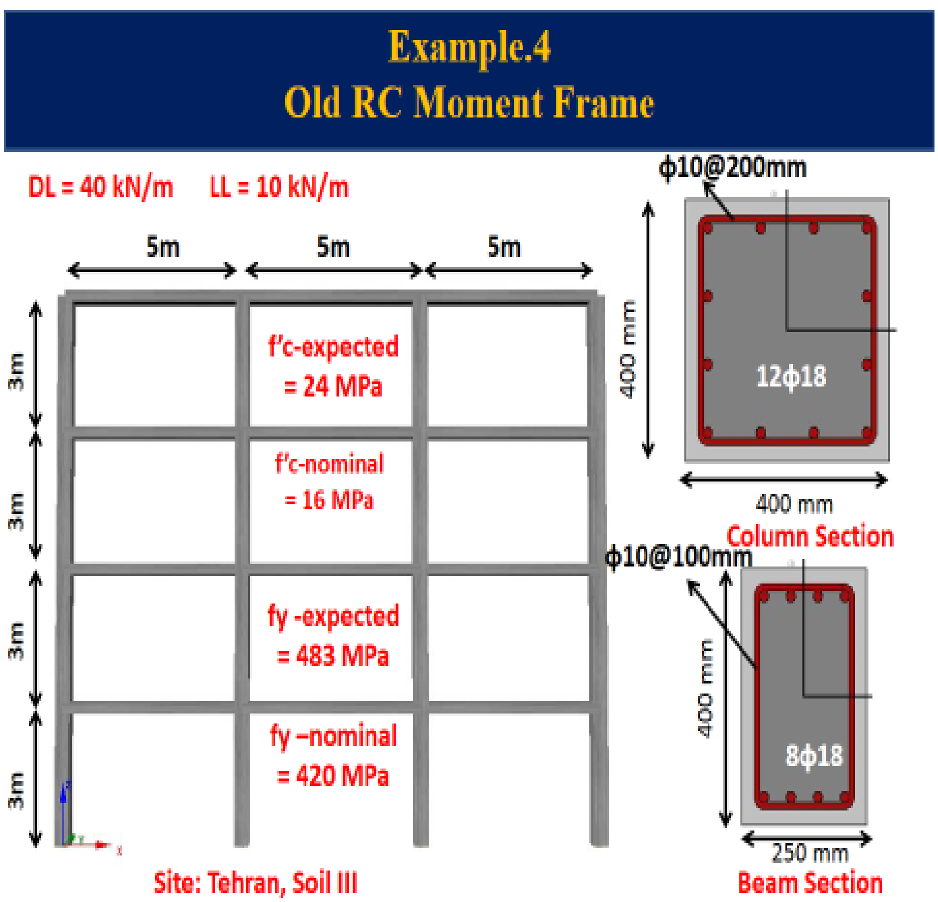
Axial load = 2320 kN
Axial lower-bound capacity = PCL=2520 kN

۴. قاب بتنی قدیمی (NC)

قاب ۴ طبقه سه دهانه / محل بنا در تهران، خاک نوع ۳، دهانه‌ها ۵ متر، ارتفاع همه طبقات ۳ متر، بار مرده و زنده طبقات به ترتیب برابر ۴ و ۱ تن بر متر/ ابعاد تیرها ۲۵۰×۴۰۰ م م و ستون‌ها ۴۰۰×۴۰۰ میلی‌متر. درصد میلگرد تیرها و ستون‌ها ۲٪

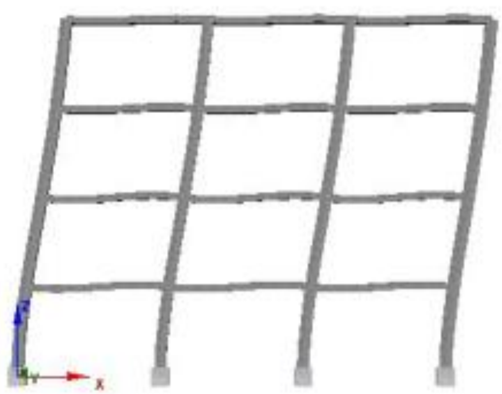
خروجی‌ها:

۱. جابجایی هدف در خطر ۱ و ۲
۲. مدل غیرخطی تیر/ مقاومت خمشی، نمودار غیرخطی
۳. مدل غیرخطی ستون/ مقاومت خمشی، نمودار غیرخطی
۴. وضعیت لولای خمیری بحرانی در تیرها در خطر ۱ و ۲
۵. وضعیت لولای خمیری بحرانی در ستون‌ها در خطر ۱ و ۲
۶. نمودار رانش
۷. نمودار دوران تیر برحسب جابجایی بام
۸. نمودار دوران تیر برحسب جابجایی بام

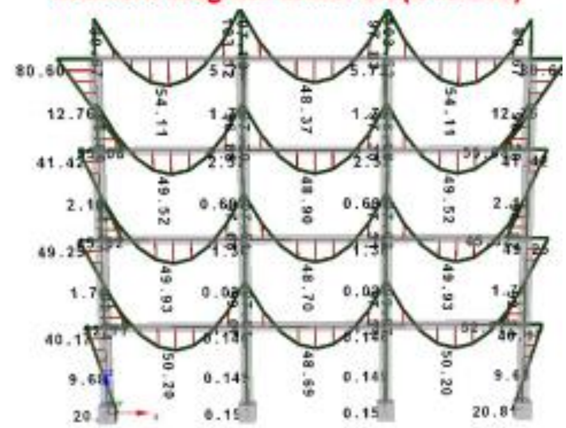


Example.4

1st mode period = 0.87 s

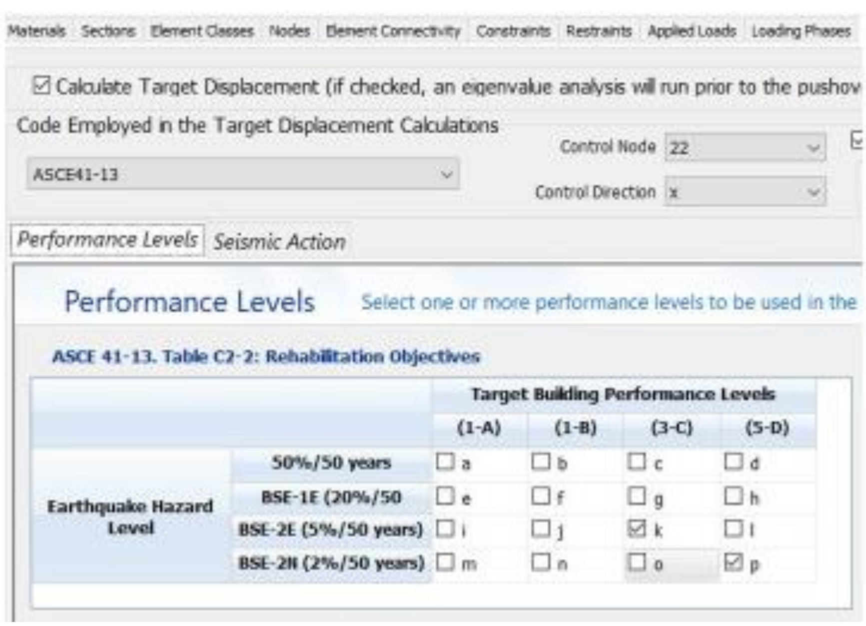


Moment diagram under 1.1(D+0.25L)



Example.4

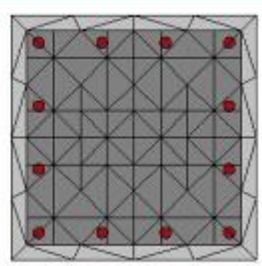
Seismic hazard in ASCE 41-13 differs with that in Code 360. To comply with Code 360, BSE-2E should be defined to be equal to the Level I hazard (10%-50 years).



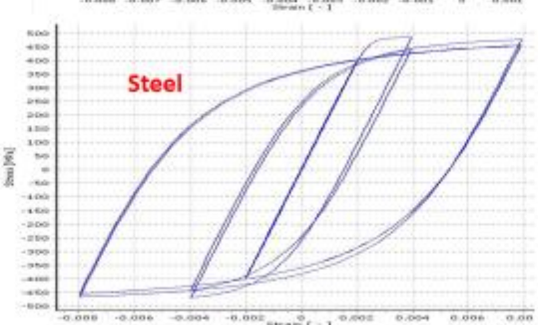
Example.4

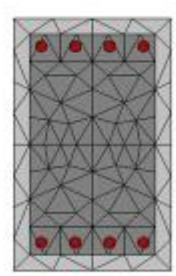
Note that these curves are from ASCE 41 which are similar to ASCE 7. target spectra in Code 360 is similar to that in Standard 2800. the target spectra of Standard 2800 and ASCE 7 differs a little, especially during longer periods

Example.4



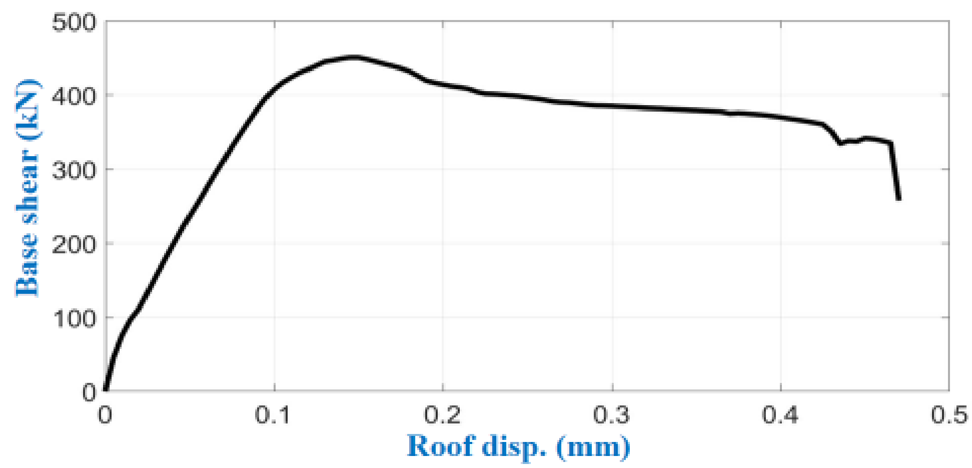
150 fibers for column section



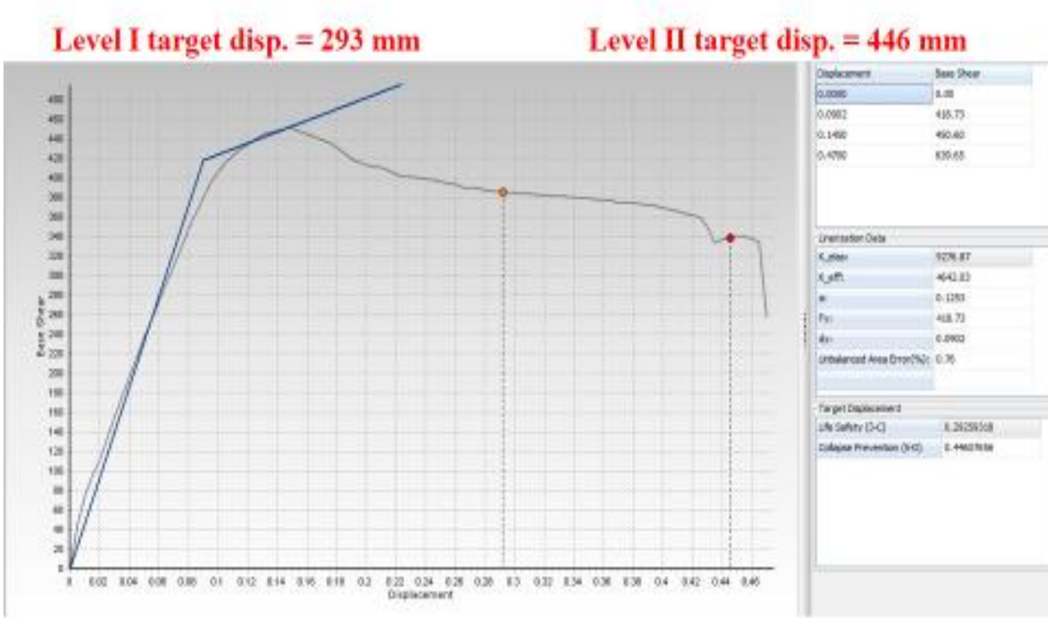


150 fibers for beam section

Example.4

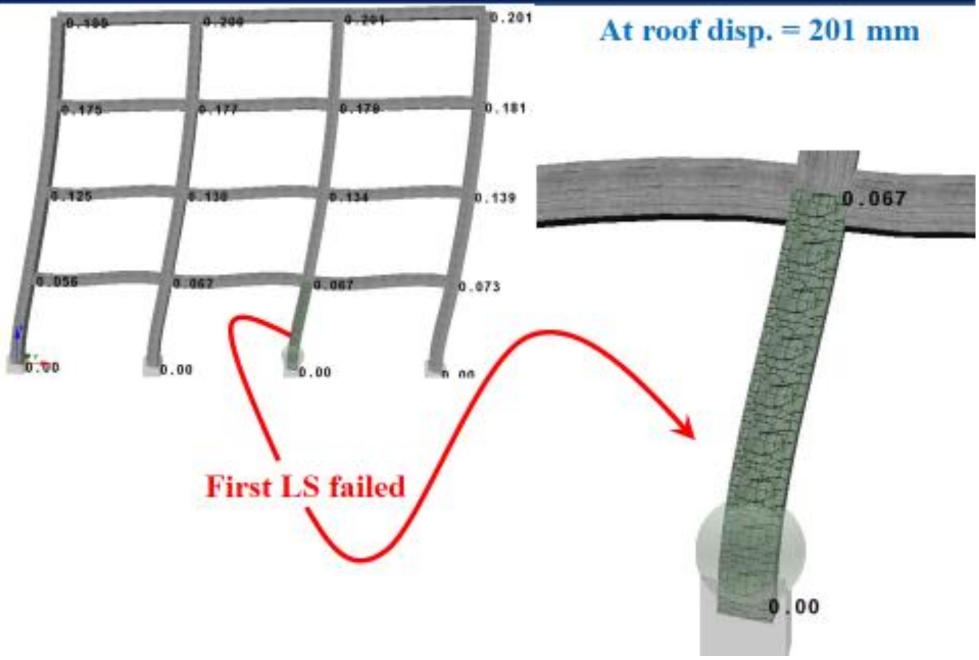


Example.4



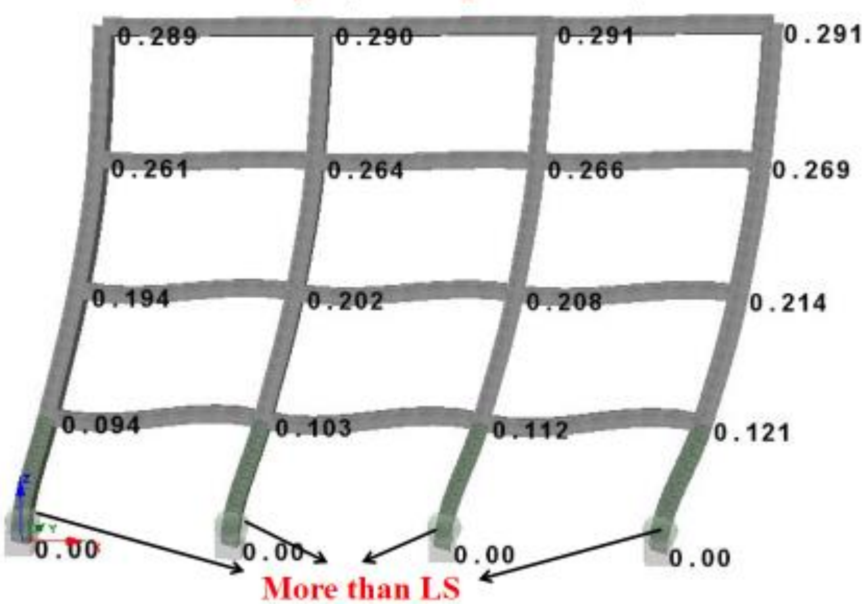
Example.4

At roof disp. = 201 mm

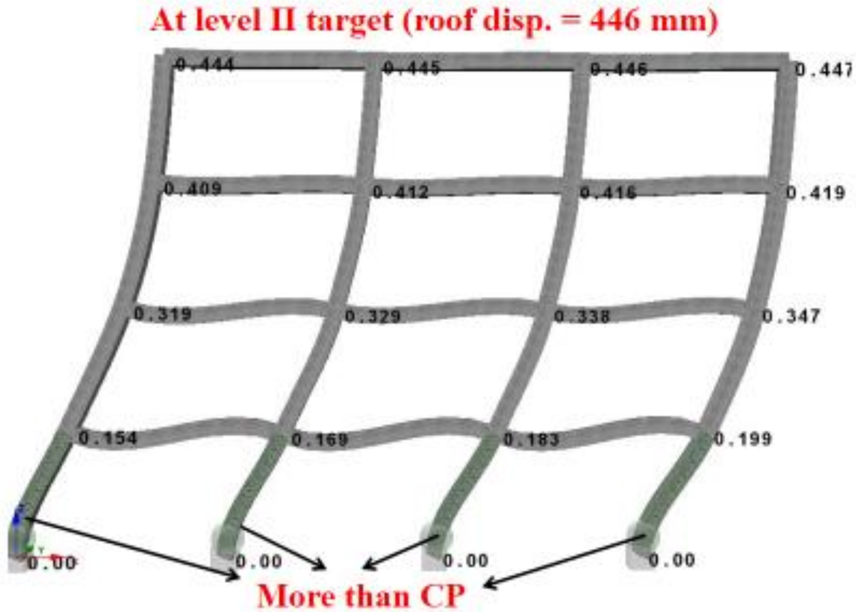


Example.4

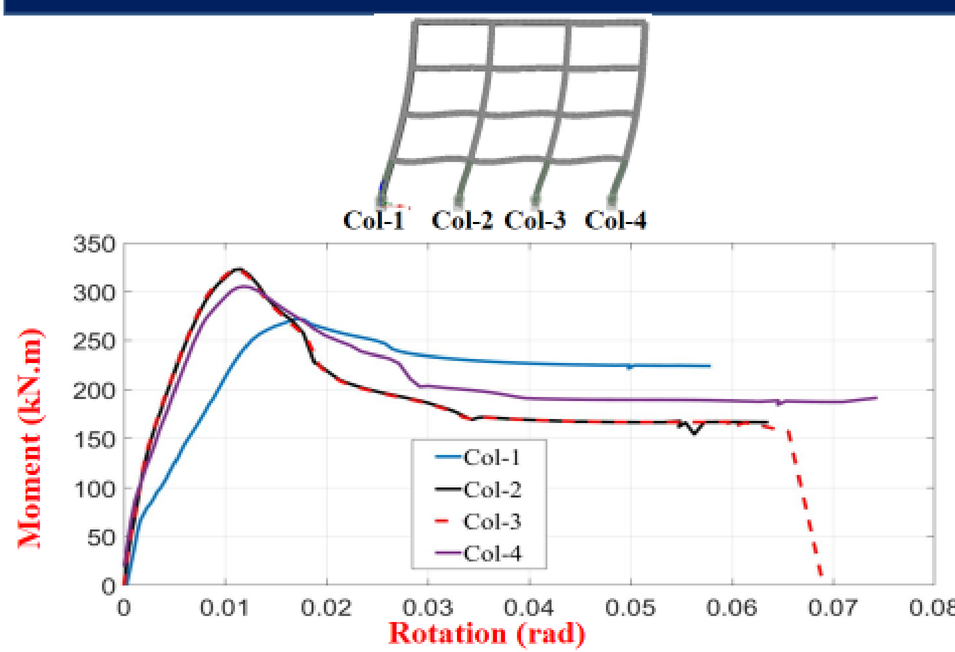
At level I target (roof disp. = 291 mm)

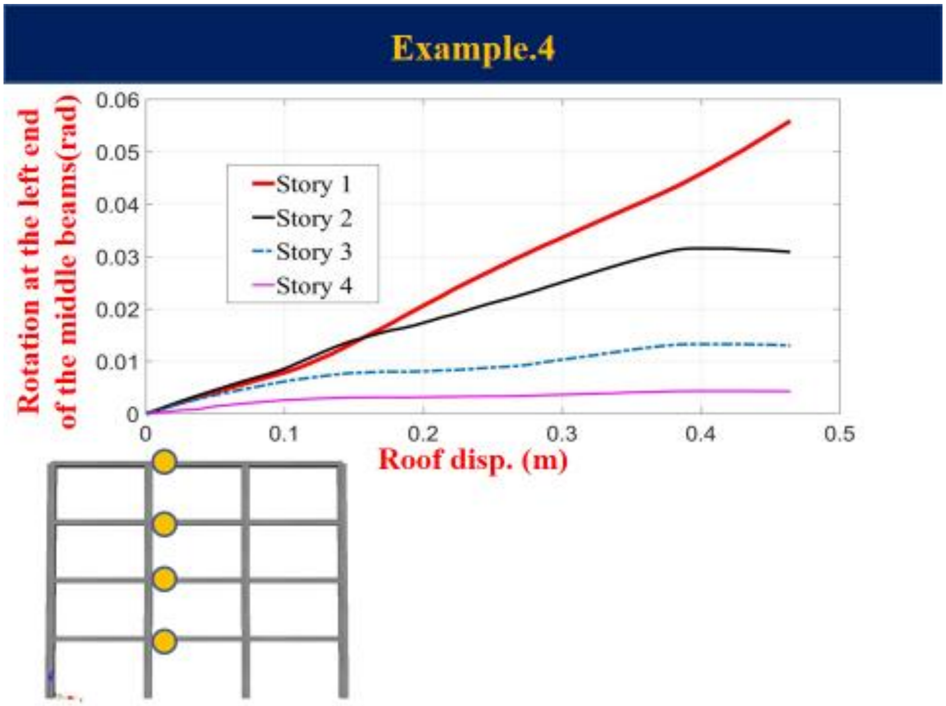


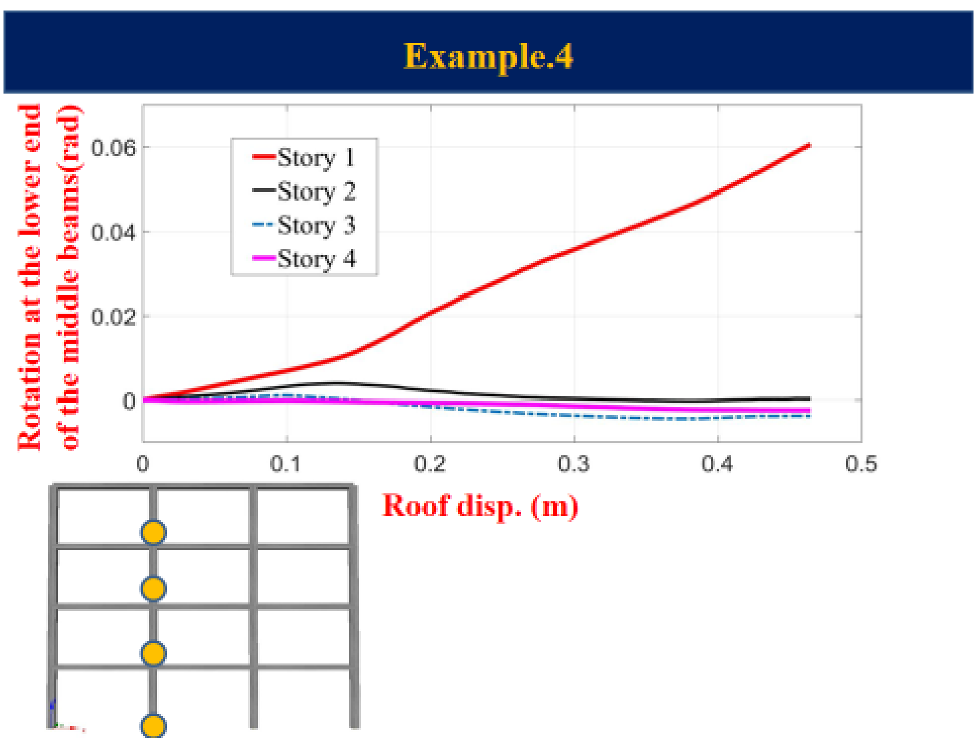
Example.4



Example.4




Example.4
Rotation at the left end of the middle beams(rad)
Roof disp. (m)
Story 1
Story 2
Story 3
Story 4
0.06
0.05
0.04
0.03
0.02
0.01
0
0.1
0.2
0.3
0.4
0.5


Example.4
Rotation at the lower end of the middle beams(rad)
Roof disp. (m)
Story 1
Story 2
Story 3
Story 4
0.06
0.04
0.02
0
0.1
0.2
0.3
0.4
0.5

۵. قاب بتنی قدیمی (NC) / تقویت با پلیمر مسلح (اف آر پی)

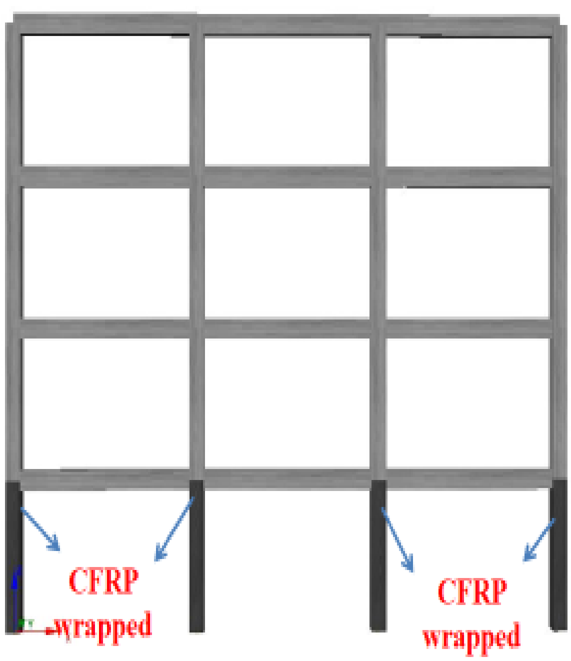
ستون‌های نمونه ۴ با پلیمر تقویت شوند

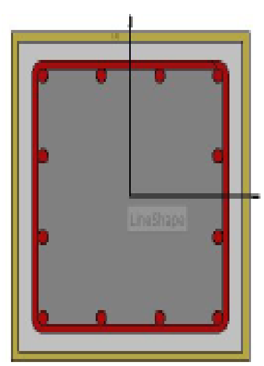
خروجی‌ها:

۱. جابجایی هدف در خطر ۱ و ۲
۲. مدل غیرخطی تیر/ مقاومت خمشی، نمودار غیرخطی
۳. مدل غیرخطی ستون/ مقاومت خمشی، نمودار غیرخطی
۴. وضعیت لولای خمیری بحرانی در تیرها و ستون‌ها در خطر ۱ و ۲
۵. نمودار رانش

Example.5
Old RC Moment Frame with FRP

Everything is Exactly similar to Example 4 except that the columns at the first story are wrapped with 3 layers of CFRP

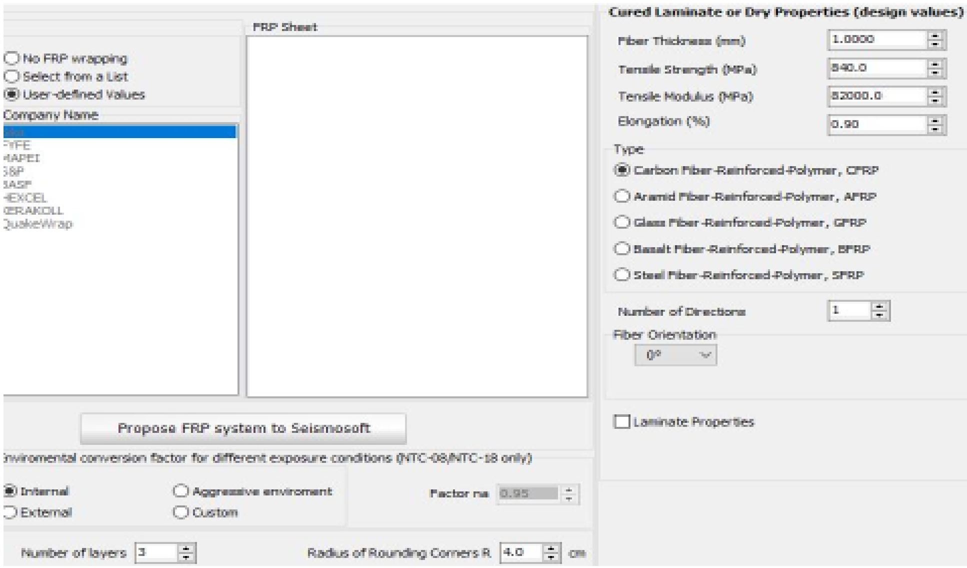




CFRP wrapped column

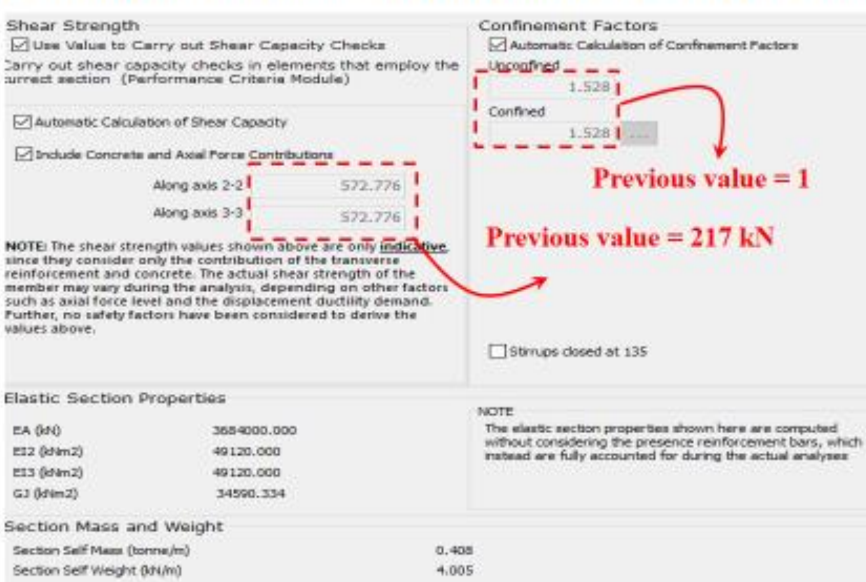
Example.5

Properties of the wrapped CFRP

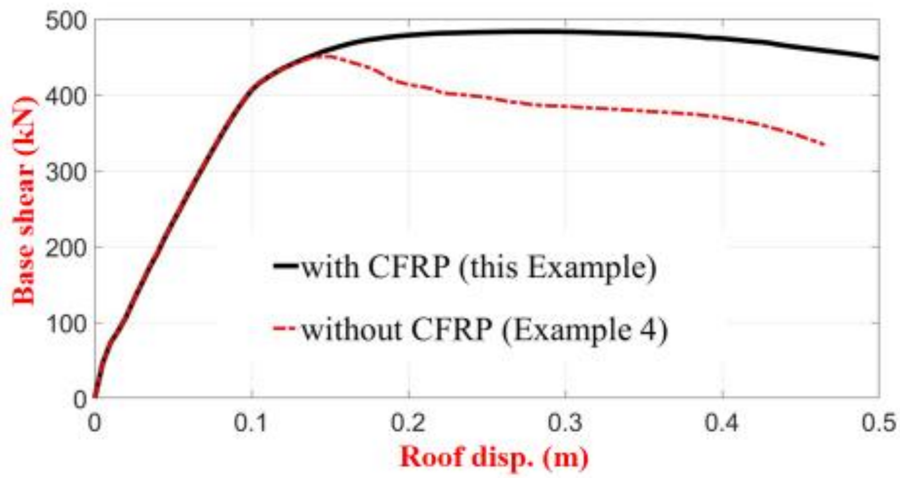


Example.5

CFRP would improve confinement and shear capacity of the section



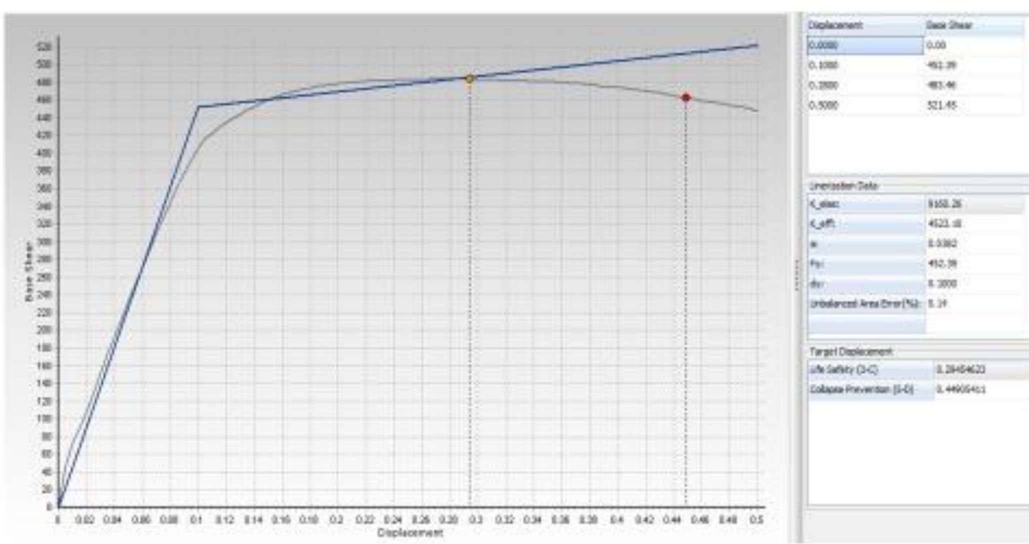
Example.5



Example.5

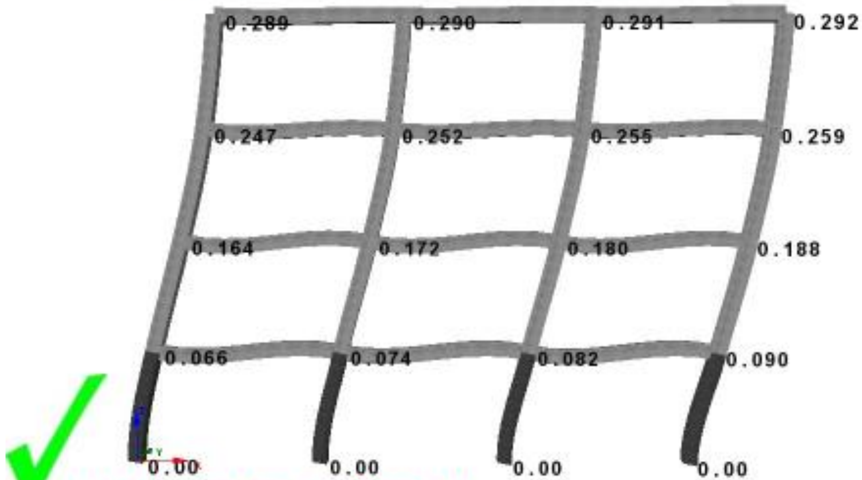
Level I target disp. = 295 mm

Level II target disp. = 449 mm



Example.5

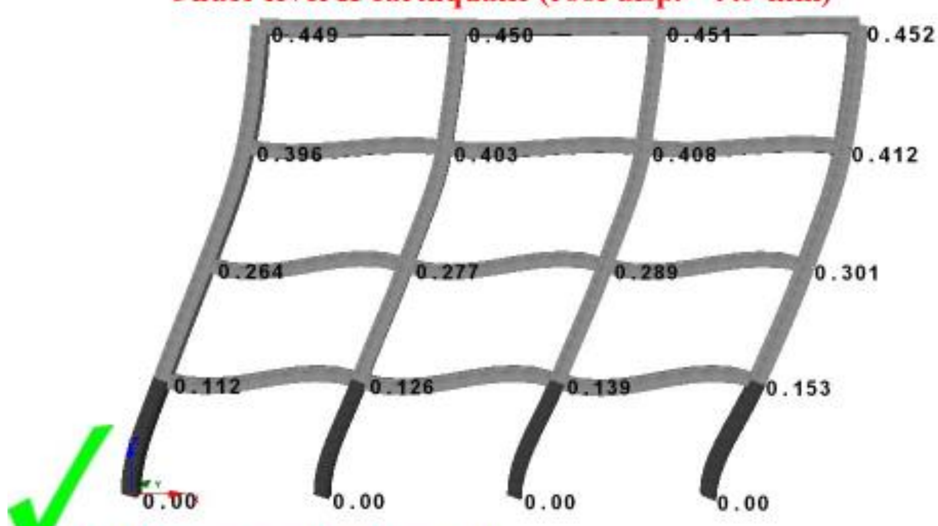
Under level I earthquake (roof disp.= 292 mm)



All elements are less than LS

Example.5

Under level II earthquake (roof disp.= 449 mm)



All elements are less than CP

۶. قاب بتنی قدیمی (NC) / تقویت با دیوار برشی

نمونه شماره ۴ که با یک دیوار برشی مذکور در نمونه ۲ تقویت‌شده

خروجی‌ها:

۱. جابجایی هدف در خطر ۱ و ۲

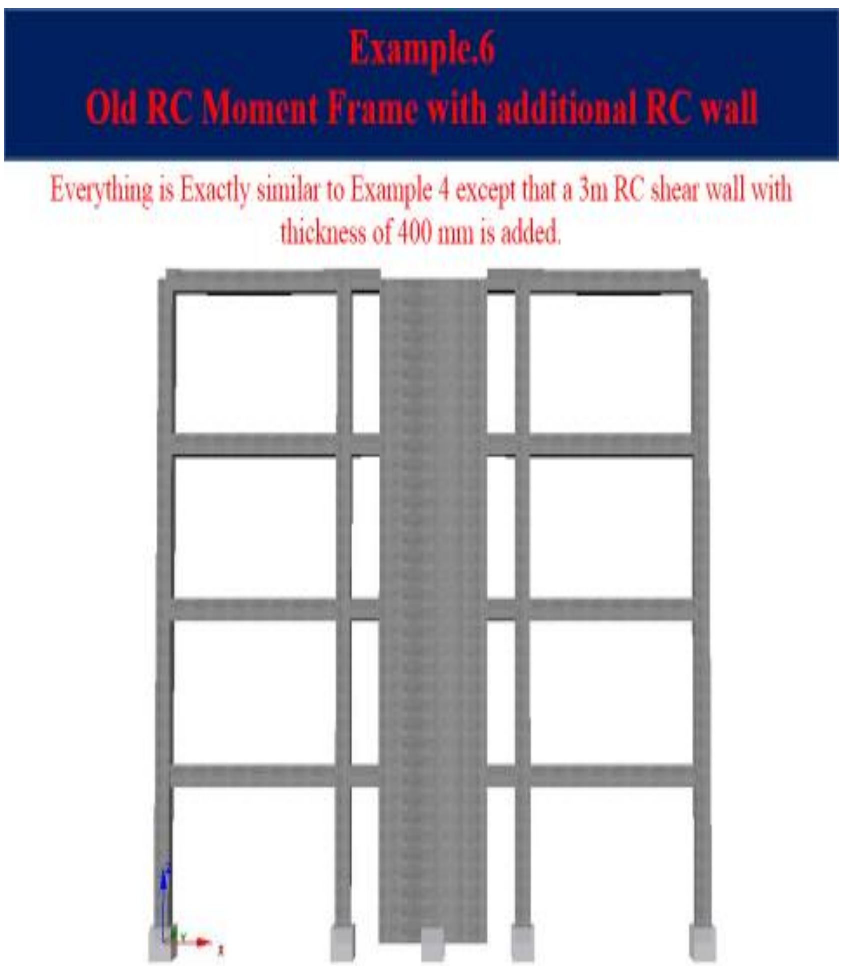
۲. وضعیت لولای خمیری بحرانی در تیرها در خطر ۱ و ۲

۳. وضعیت لولای خمیری در تیرهای متصل به دیوار برشی در خطر ۱ و ۲

۴. وضعیت لولای خمیری بحرانی در ستون‌ها در خطر ۱ و ۲

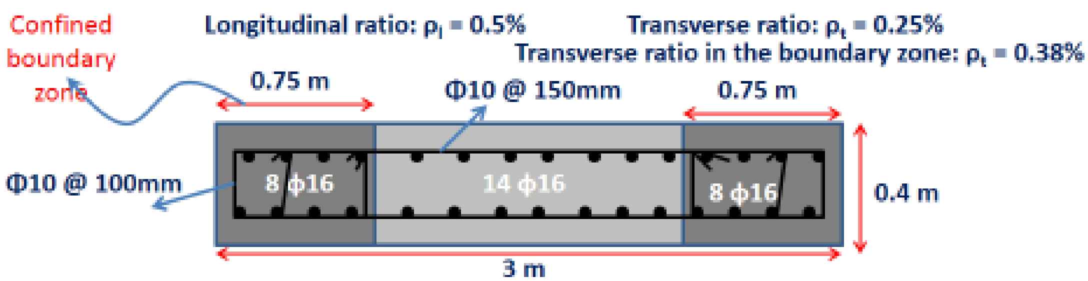
۵. نمودار رانش

۶. جابجایی و گریز طبقات

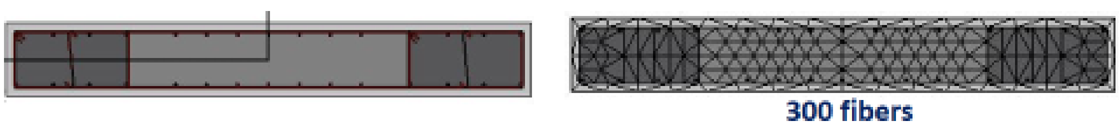


Example.6

The added RC wall is similar to that used in Example 2 for the steel frame with khorjini connections.

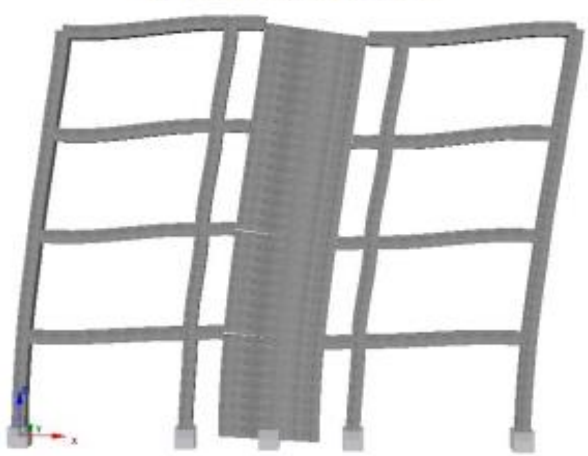


Modeled section in SStruct



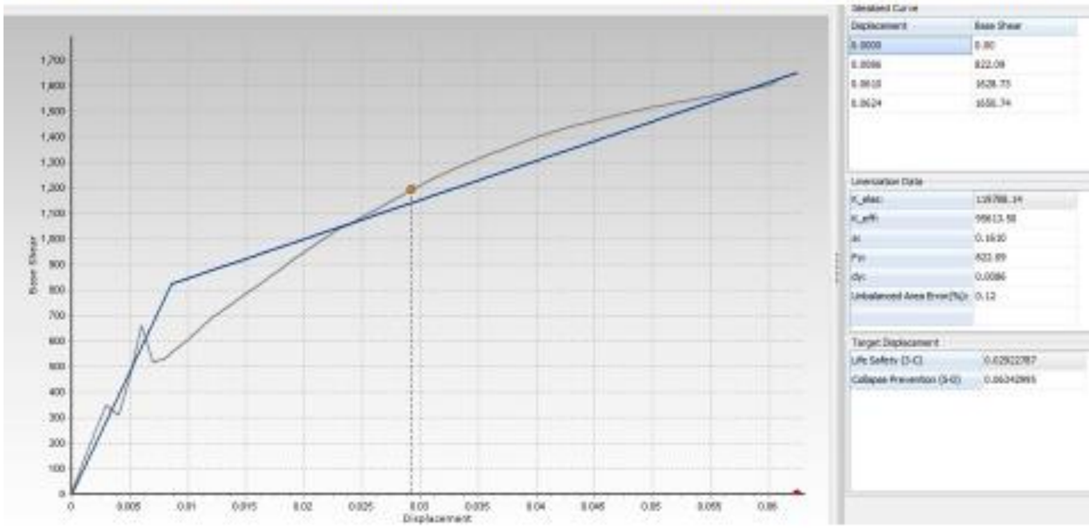
Example.6

1^{st} mode period = 0.208 s



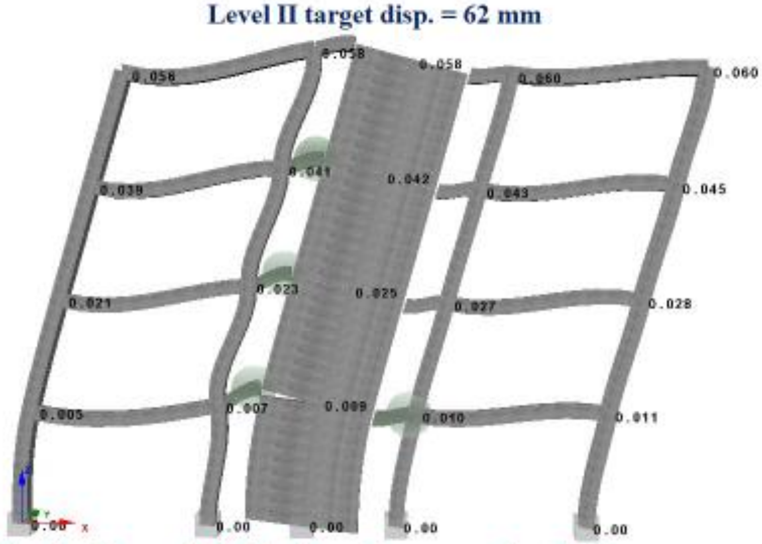
Example.6

Level I target disp. = 29 mm **Level II target disp. = 62 mm**



Example.6

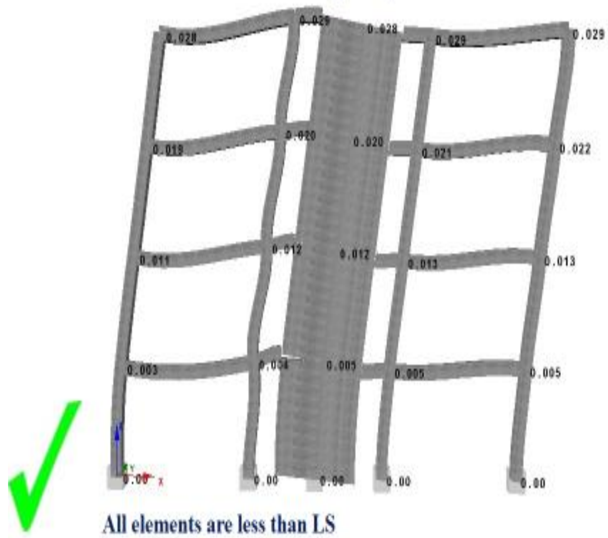
Level II target disp. = 62 mm



The short beams failed in shear (shear capacity check)

Example.6

Level I target disp. = 29 mm



All elements are less than LS

Example.6

این مثال با دیوار 5 متری در دهانه وسط
تکرار شود

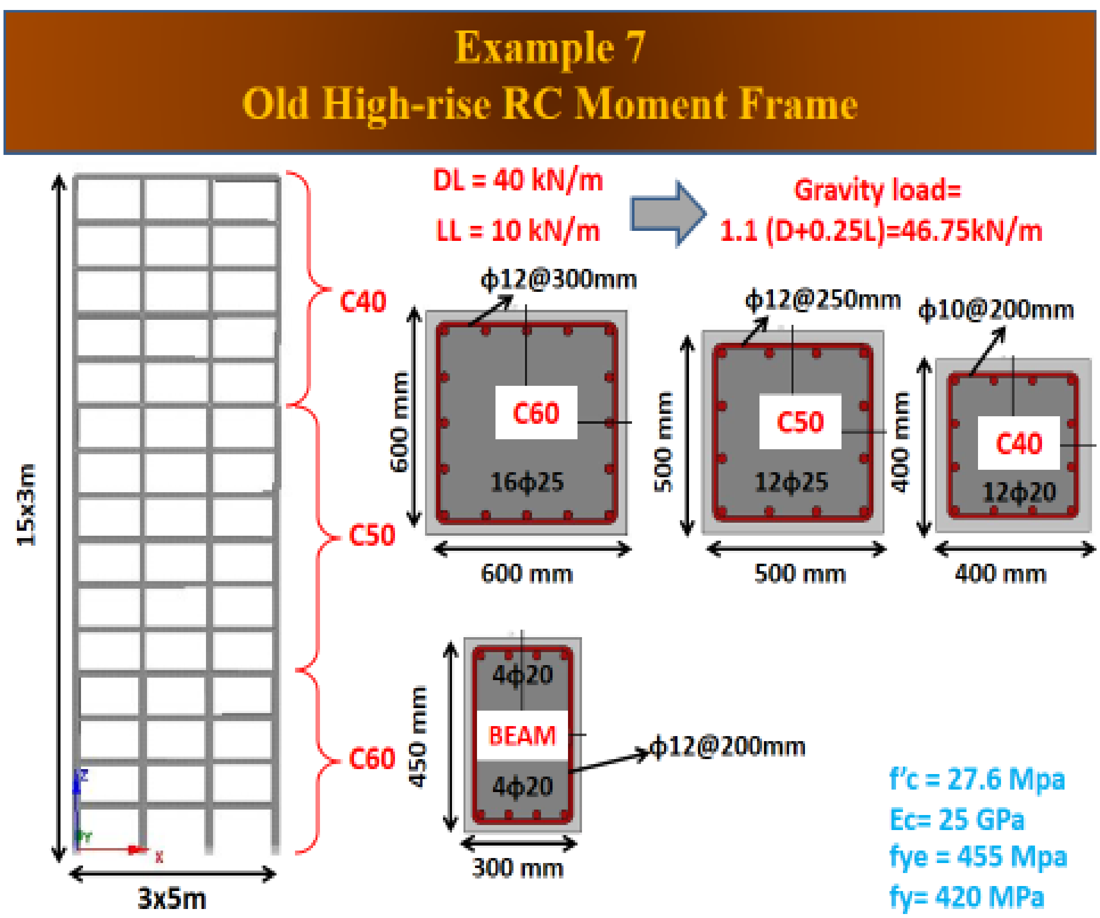
۷. قاب بتنی قدیمی (NC)

قاب بتنی سه دهانه ۱۵ طبقه

محل بنا در تهران، خاک نوع ۳، دهانه‌ها ۵ متر، ارتفاع همه طبقات ۳ متر، بار مرده و زنده طبقات به ترتیب برابر ۴ و ۱ تن بر متر/ ابعاد تیرها ۳۰۰×۴۵۰ م م و ستون‌های طبقات ۱ تا ۴: ۶۰۰×۶۰۰، طبقات ۵ تا ۱۰ مقطع ۵۰۰×۵۰۰ و طبقات ۱۱ تا ۱۵ مقطع ۴۰۰×۴۰۰ میلی‌متر. درصد میلگرد تیرها و ستون‌ها ۲٪.

خروجی‌ها:

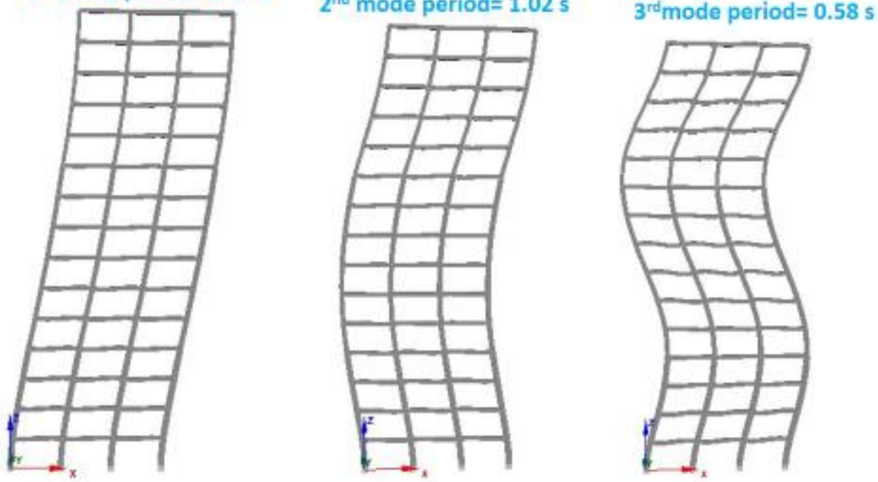
۱. جابجایی هدف در خطر ۱ و ۲
۲. وضعیت لولای خمیری در تیرهای بحرانی در خطر ۲
۳. وضعیت لولای خمیری در ستون‌های بحرانی در خطر ۲
۴. نمودار رانش
۵. جابجایی و گریز طبقات



Example 7

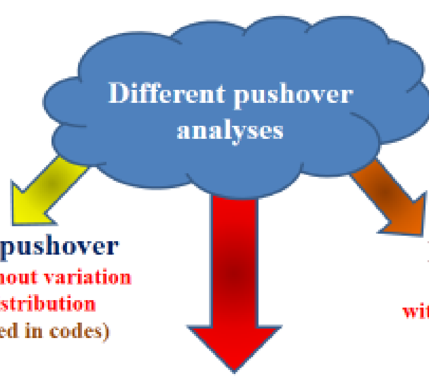
Uncracked Periods

All beams and columns are FB elements with stress recovery



Example 7

As this is a high-rise building, we prefer to run an adaptive pushover analysis



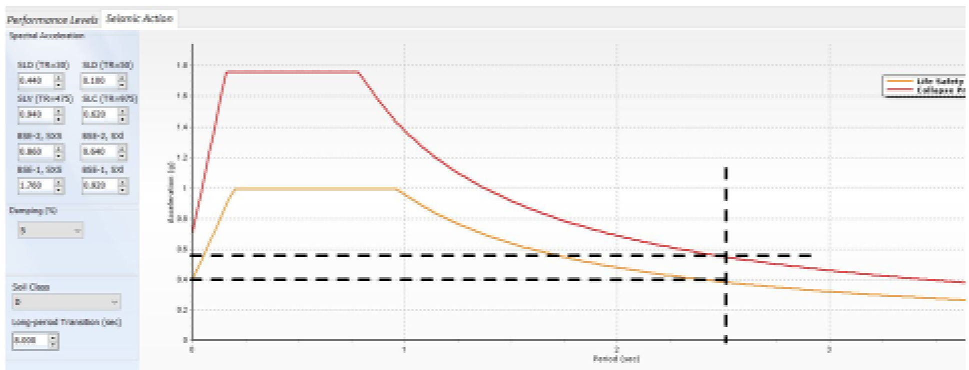
Conventional pushover
Only first mode without variation of lateral load distribution
(the procedure used in codes)

Modal pushover
Higher mode effects without variation of lateral load distribution

Adaptive pushover
Variation of lateral load distribution + higher mode effects

Example 7

As Sstruct spectrum is based on ASCE 41 (similar to ASCE 7), its shape a little differs from Standard 2800 spectrum, especially in longer periods. So the target spectrum in Sstruct should be defined such that Sa near the effective period of the building match with that in Standard 2800



Sa value in Te should match with standard 2800

Example 7

In this example we would manually define rotational capacities of beams because automatic calculation of Sstruct is based on column tables even for beams

For beams and columns from the equation (4.29) of D.Biskinis (2007):

$$\theta_y = \frac{M_y L_s}{3EI_{eff}}$$ ⇒ EI=56250 kN.m² ⇒ Eleff=0.3x56250=16875 kN.m²

My=210 kN.m (from Sstruct element result)

LS = 0.5L=2.5 m

θy=0.0104 rad

For beams:

Shear capacity = 150 kN

Demand = 2Mp/L+V(gravity)=2x220/5+46x5/2=203 kN

Beams are controlled by shear

Also it can be seen by a preliminary analysis and check shear capacities

From ASCE 41's Table 10-7 (or Code 360):

Condition ii (shear dominate)

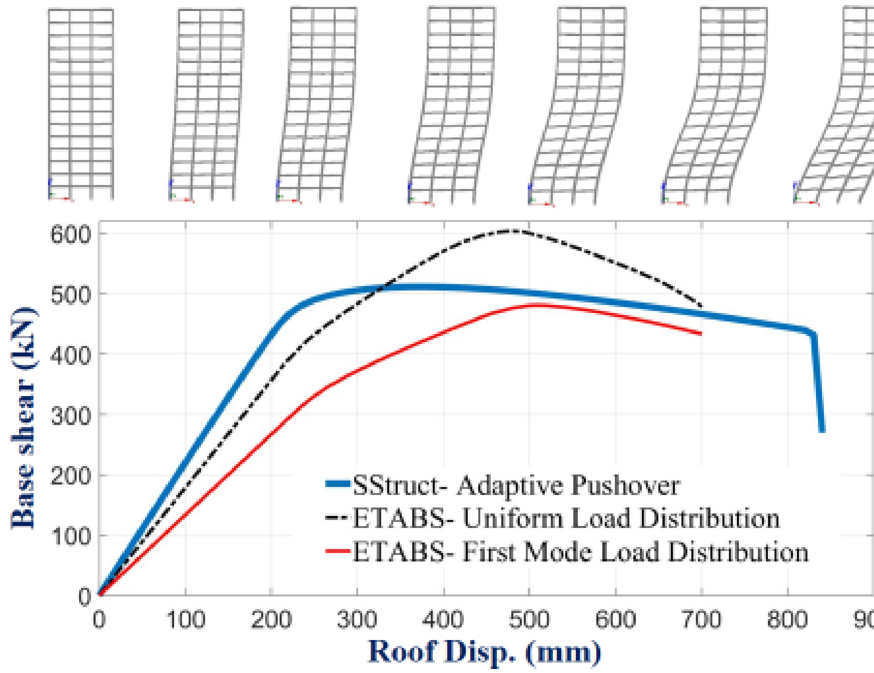
S=250>d/2 ⇒ θp-LS=0.005, θp-CP=0.01 ⇒

Total rotation-LS=0.0104+0.005=0.015 rad

Total rotation-CP=0.0104+0.01=0.02 rad

Sstruct obtained rotational capacity of beam similar to columns which is not conservative: per Sstruct: total rot-LS=0.04 rad and total rot-CP=0.048 rad

Example 7

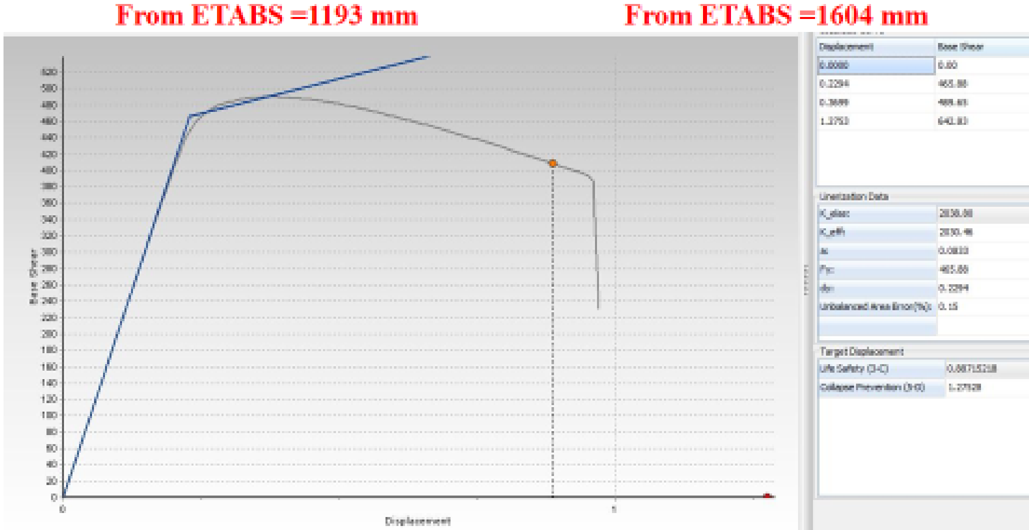


Example 7

The difference between ETABS and Sstruct is due to the different estimated period. From Sstruct T1=2.47s but from ETABS T1=3.96s. Period from Sstruct is more accurate as it accounts for reinforcements effects on stiffness.

Seismic hazard I target disp. = 887 mm
From hand calculation = 854 mm
From ETABS =1193 mm

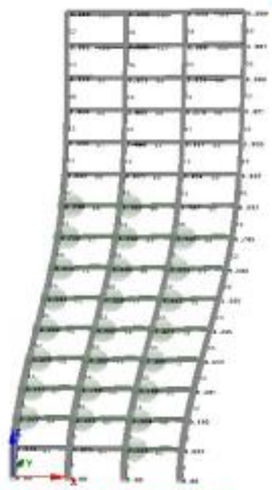
Seismic hazard I target disp. = 1275 mm
From hand calculation = 1280mm
From ETABS =1604 mm



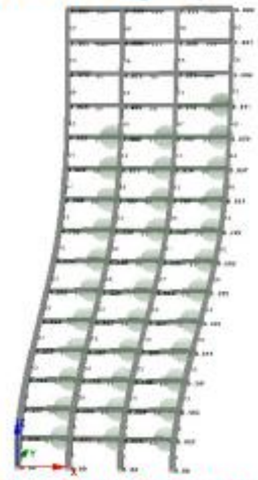
Example 7

Seismic hazard I, At roof disp. = 887 mm

Actually only flexure failures should be considered as effect of shear is already considered in rotational capacity



Beyond LS criterion (shear and flexure)

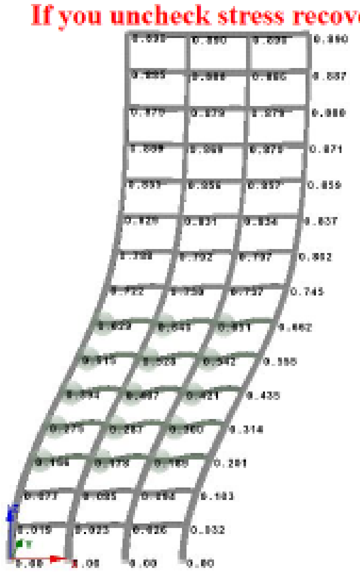


Where shear demands exceeds shear capacity

Example 7

Seismic hazard I, At roof disp. = 887 mm

If you uncheck stress recovery



Beyond LS criterion (shear and flexure)

If stress recovery is unchecked, shear demands would be smaller than reality and designer might decide to use flexure control acceptance criteria. So you need to use stress recovery to capture shear failure…at least at the preliminary analysis.

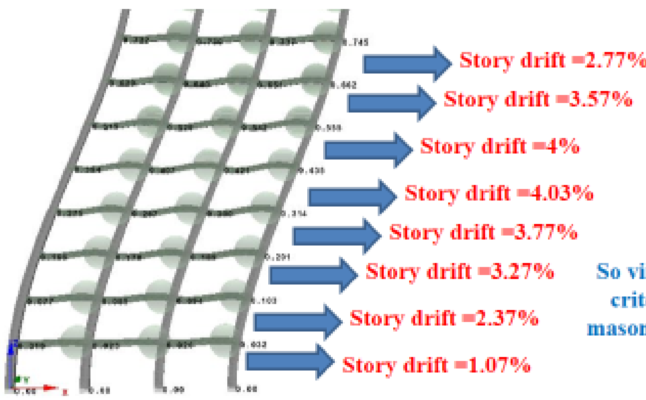
No shear demand exceeds shear capacity..!

All beams failed in flexure (not shear)

Example 7

In addition to structural criteria, non-structural criteria should also be satisfied

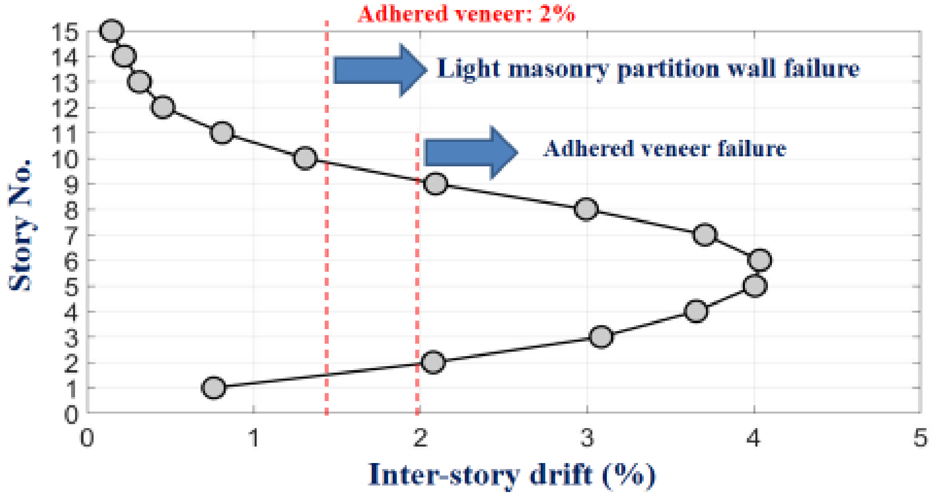
For example at level I target disp., inter-story drift of many stories are quite high.
Per ASCE 41-13, to satisfy LS performance for non-isolated masonry partition walls, inter-story drift should be less than 1%.



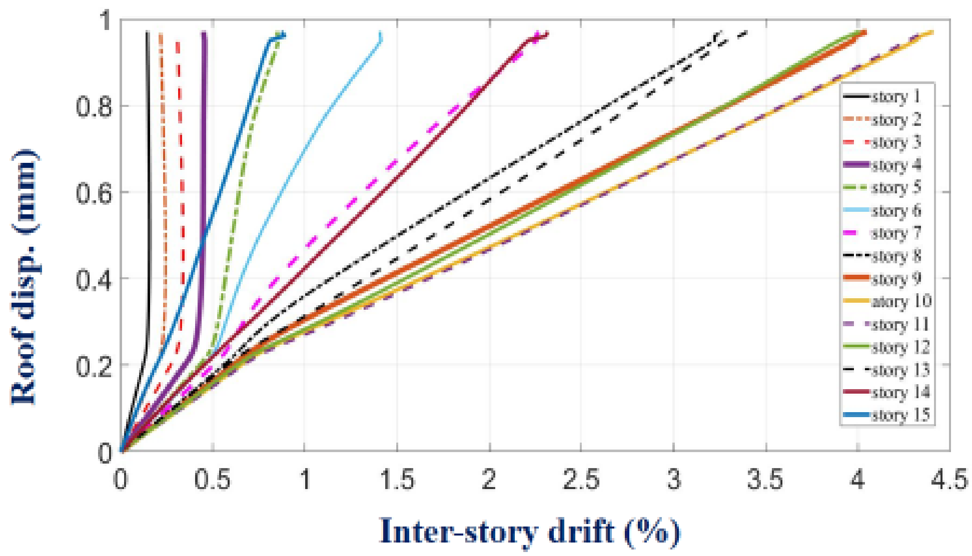
So virtually in all stories LS criterion for non-isolated masonry partition walls is not satisfied.

Example 7

Per Code 360, LS drift for different non-structural components are as follows,
Non-isolated heavy masonry partition wall: 1%
Non-isolated light masonry partition wall: 1.5%
Non-isolated precast panels wall: 2%
Non-isolated exterior masonry wall: 0.8%
Adhered veneer: 2%



Example 7

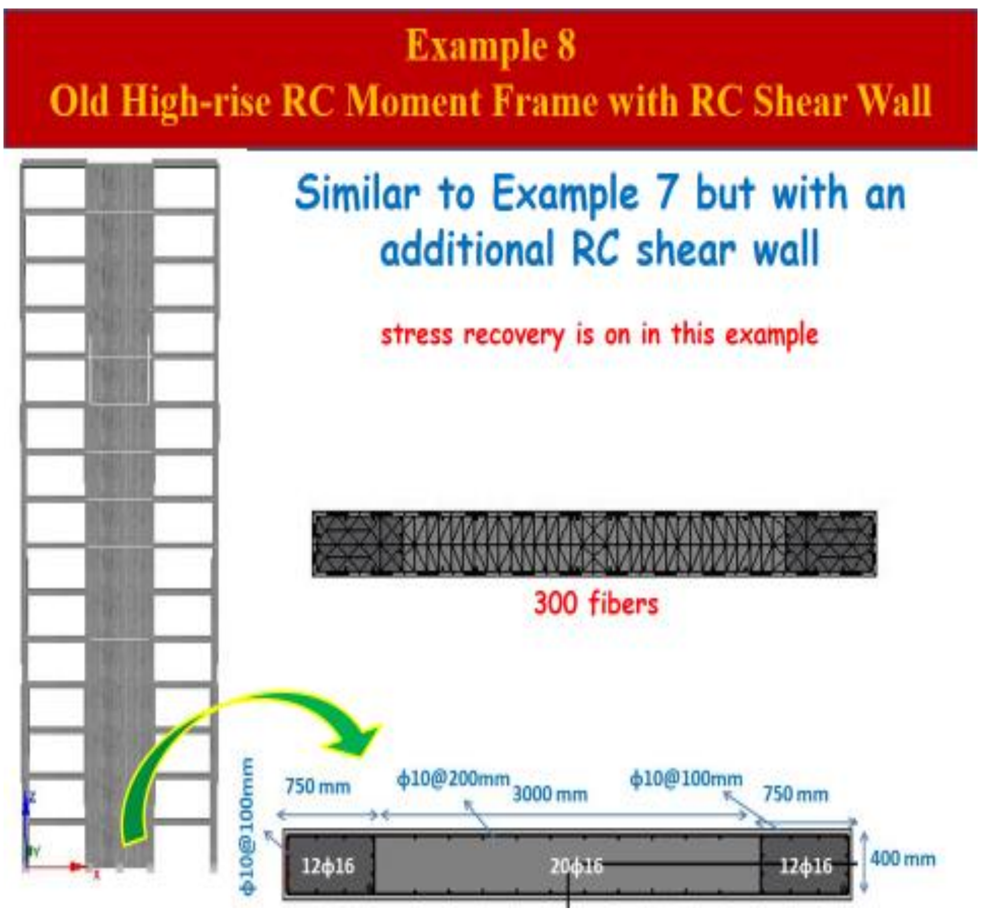

Roof disp. (mm)
Inter-story drift (%)
story 1
story 2
story 3
story 4
story 5
story 6
story 7
story 8
story 9
atory 10
story 11
story 12
story 13
story 14
story 15

۸. قاب ۱۵ طبقه شماره ۷ / تقویت با دیوار برشی

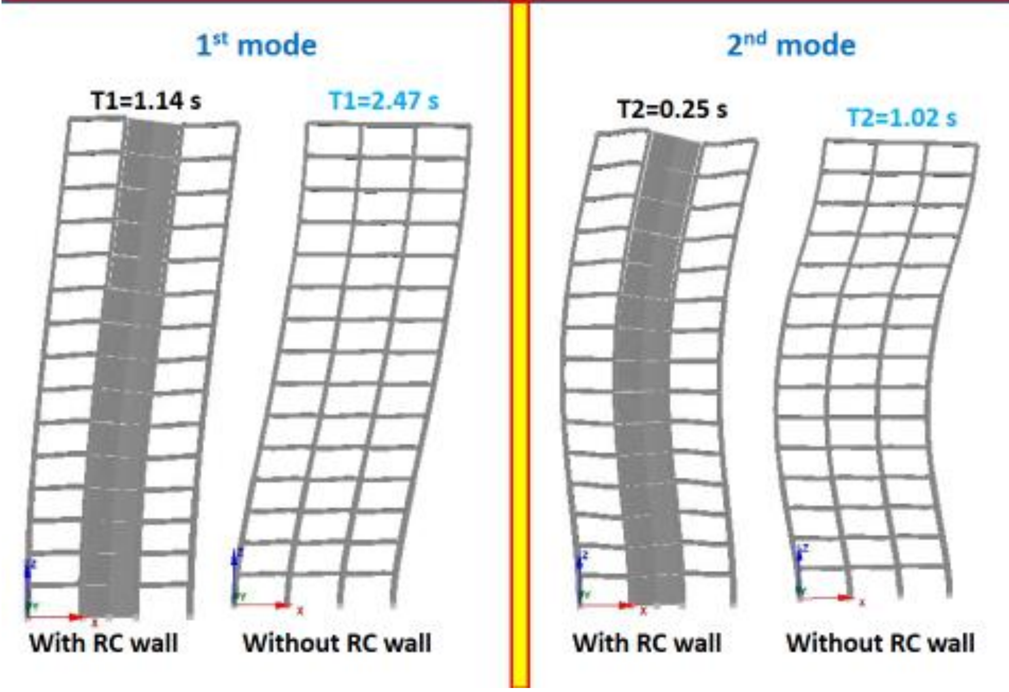
نمونه شماره ۷ بعلاوه یک دیوار برشی به طول ۵ متر، کلفتی ۰/۴ متر و میلگرد طولی ۰/۵٪.

خروجی‌ها:

۱. جابجایی هدف در خطر ۱ و ۲
۲. وضعیت لولای خمیری در تیرهای بحرانی در خطر ۲
۳. وضعیت لولای خمیری در ستون‌های بحرانی در خطر ۲
۴. نمودار رانش
۵. جابجایی و گریز طبقات
۶. نمودار لولای خمشی در دیوار برشی برحسب جابجایی بام
۷. درصورتی‌که دیوار برشی بر روی دو شمع تکیه کرده باشد نیروهای فشاری و کششی در این شمع‌ها در حالت اوج

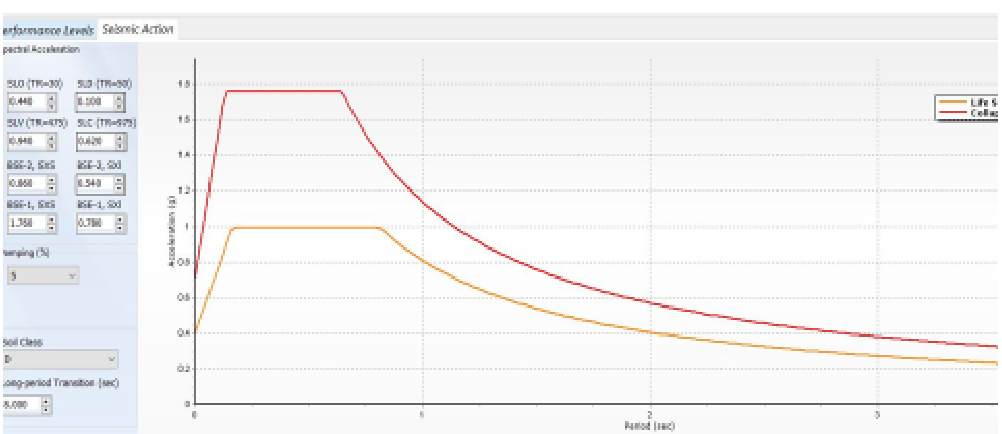


Example 8



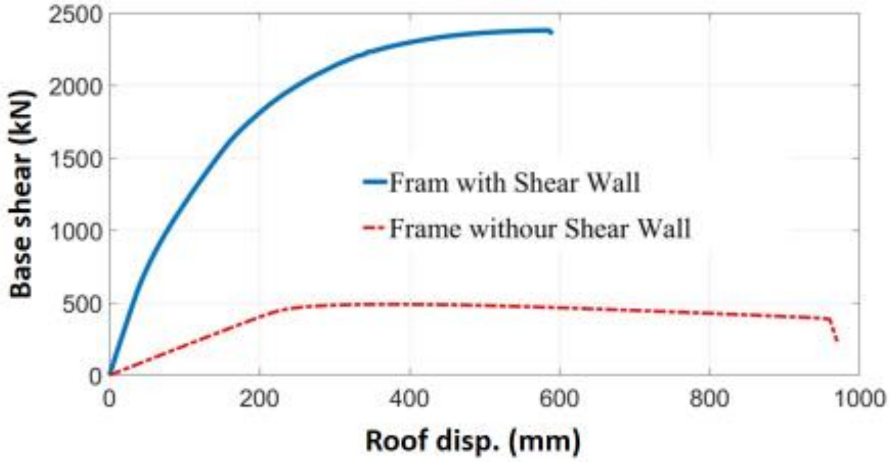
Example 8

Target spectra should be adjusted to match with ST 2800 in
T= 1.15 s (roughly)



Example 8

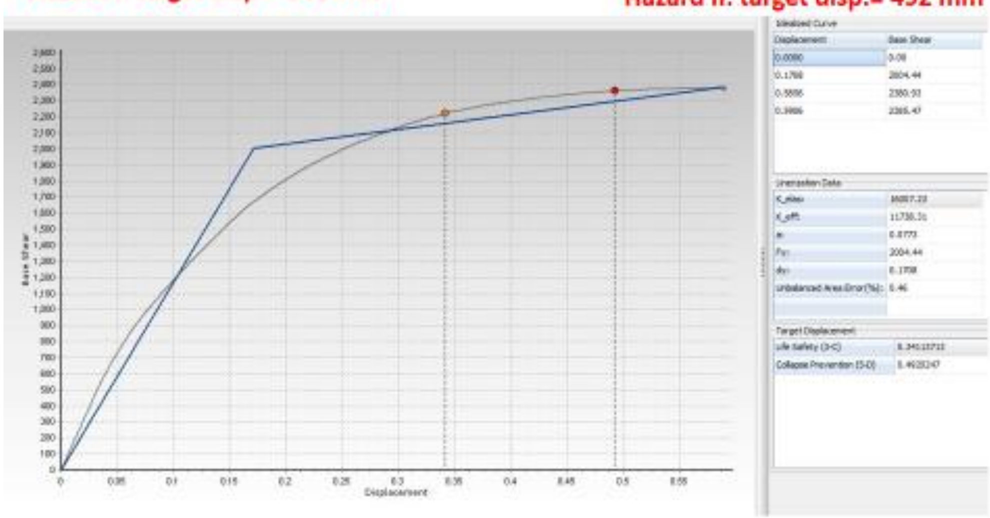
Is the foundation able to support the increased base shear and the related axial loads at the wall boundaries?!



Example 8

Hazard I: target disp.= 341 mm

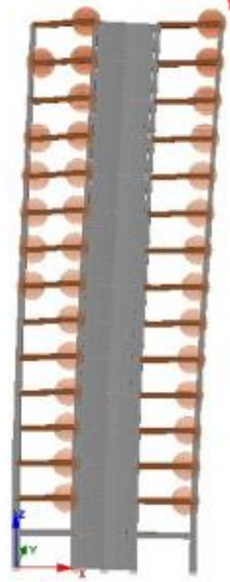
Hazard II: target disp.= 492 mm



Example 8

Hazard I: target disp.= 341 mm

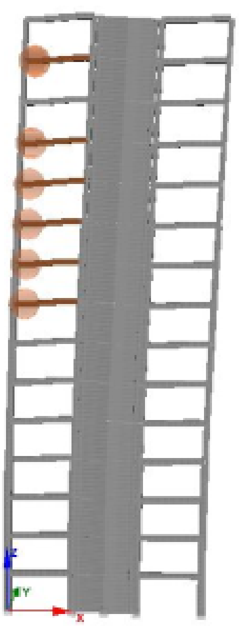
Are the beams still shear control?
yes



Where shear capacity is smaller than shear demand

Example 8

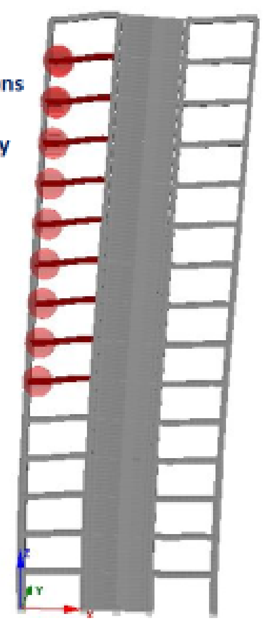
Hazard I: target disp.= 341 mm



Beams with rotations higher than LS rotational capacity (0.015 rad)

Hazard II: target disp.= 492 mm

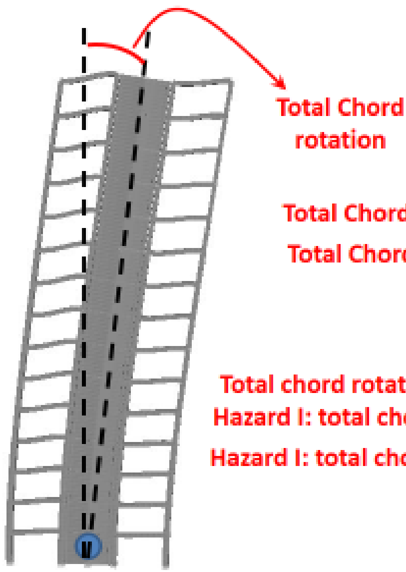
Beams with rotations higher than CP rotational capacity (0.02 rad)



We should consider only flexural failures not shear failures as we considered shear failures by reducing rotational capacities of shear-dominate beams

Example 8

For shear walls, it is better to check chord rotations by hand calculations as Sstruct gives story-by-story chord rotation....chord rotation of each element not the whole wall



Total Chord rotation capacity-LS=0.0089 rad

Total Chord rotation capacity-CP=0.018 rad

Total chord rotation ≈roof disp./wall height

Hazard I: total chord rotation = 0.341/(3x14+1.5)=0.0078 OK.

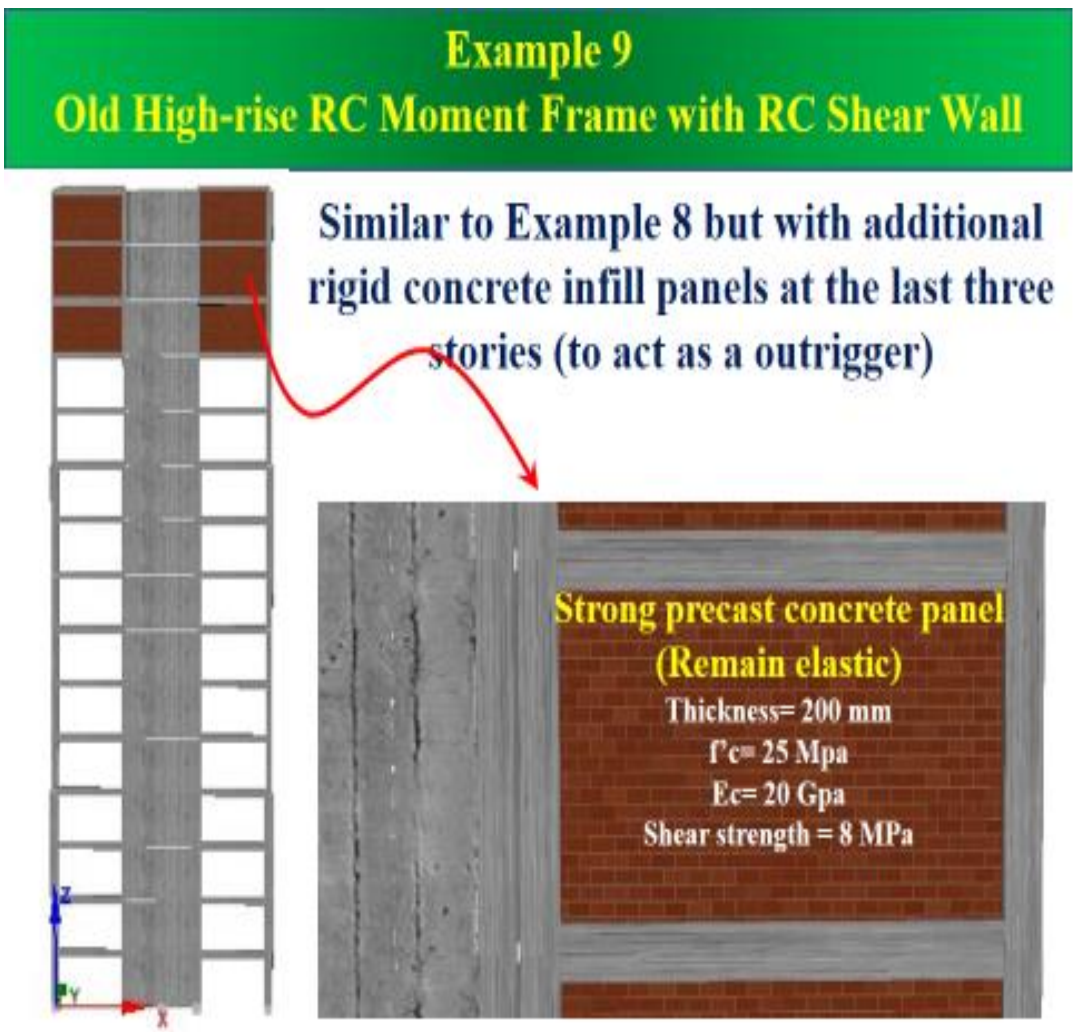
Hazard I: total chord rotation = 0.492/(3x14+1.5)=0.0113 OK.

۹. قاب ۱۵ طبقه شماره ۷ / تقویت با دیوار برشی + میان‌قاب بتنی

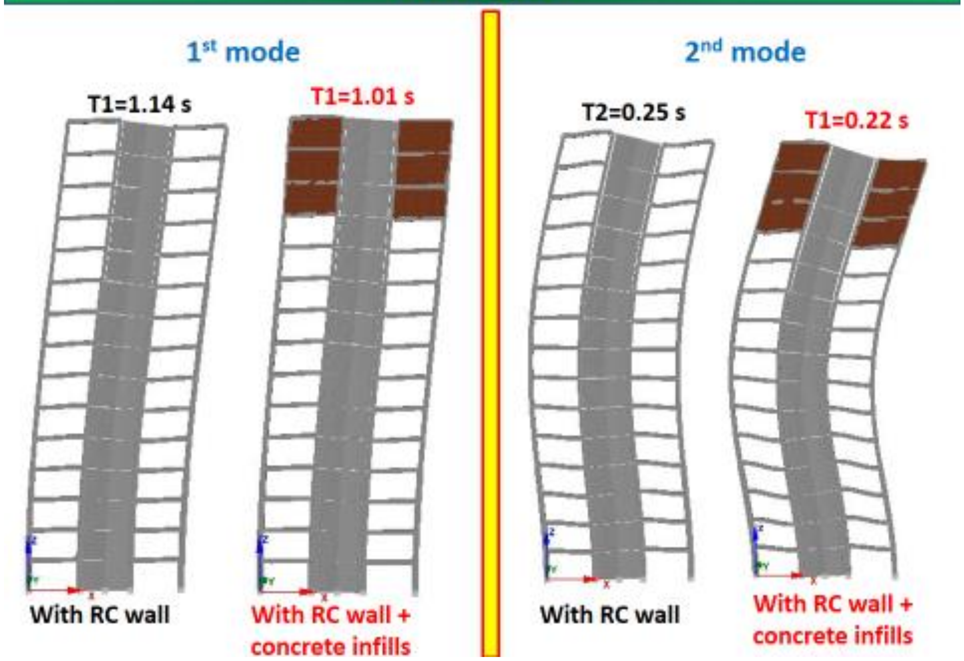
نمونه شماره ۸ بعلاوه میان‌قاب‌های بتنی به کلفتی ۲۰۰ میلی‌متر در سه طبقه آخر

خروجی‌ها:

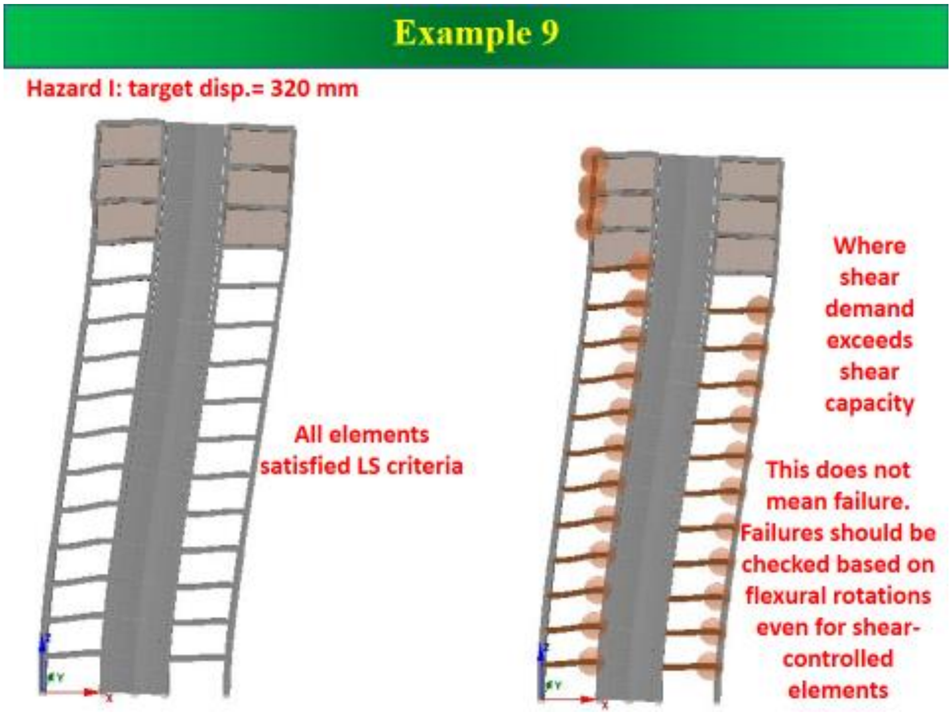
۱. جابجایی هدف در خطر ۱ و ۲
۲. وضعیت لولای خمیری در تیرهای بحرانی در خطر ۲
۳. وضعیت لولای خمیری در ستون‌های بحرانی در خطر ۲
۴. نمودار رانش
۵. جابجایی و گریز طبقات
۶. نمودار لولای خمشی در دیوار برشی در طبقات اول و دوازدهم برحسب جابجایی بام

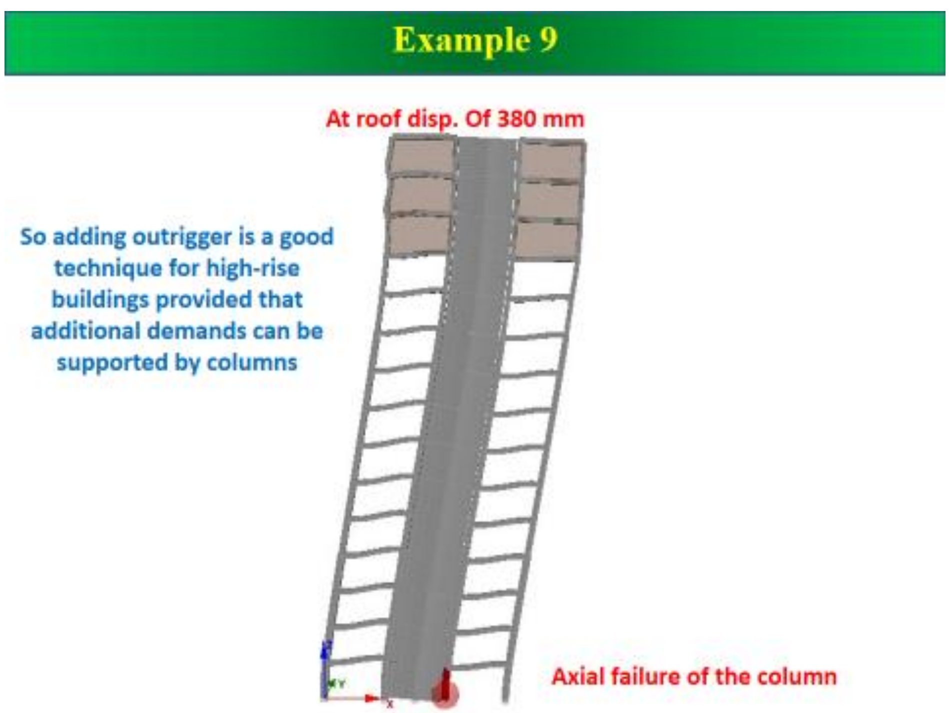


Example 9



Example 9


Example 9
Hazard I: target disp.= 320 mm
All elements satisfied LS criteria
Where shear demand exceeds shear capacity
This does not mean failure. Failures should be checked based on flexural rotations even for shear-controlled elements


Example 9
At roof disp. Of 380 mm
So adding outrigger is a good technique for high-rise buildings provided that additional demands can be supported by columns
Axial failure of the column

لازم به ذکر است که بخش نرم‌افزاری این دوره در دو گروه به شرح زیر برگزار می‌شود:

بخش	زمان برگزاری	ساعت برگزاری	مدت دوره
نرم‌افزاری	جمعه ۱۰ اسفند ۹۷ و چهارشنبه، پنجشنبه و جمعه از ۱۵ تا ۱۷ اسفند ۹۷، پنجشنبه و جمعه ۲۳ تا ۲۴ اسفند ۹۷	۹ تا ۱۶	۴۲ ساعت

برای کسب اطلاعات بیشتر به لینک زیر مراجعه کنید:

http://civil808.com/landing/pbdcourse